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MODELLING ACTIVITIES RELATED TO FLOOD DAMAGE REDUCTION — Proceedings of the Technology Transfer Workshop

ACTIVITÉS DE MODÉLISATION RELIÉES À LA RÉDUCTION DES DOMMAGES DUS AUX INONDATIONS — Comptes rendus de l'atelier sur l'échange d'informations d'ordre technologique

TECHNICAL WORKSHOP SERIES NO. 2
ÉTUDE Nº 2, SERIE DES ATELIERS TECHNIQUES

INLAND WATERS DIRECTORATE
OTTAWA, CANADA, 1980

DIRECTION GENERALE DES EAUX INTERIEURES
OTTAWA, CANADA, 1980

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PROCEEDINGS*
# of the
TECHNOLOGY TRANSFER WORKSHOP

MODELLING ACTIVITIES RELATED TO FLOOD DAMAGE REDUCTION

Hull, Quebec, May 23-25, 1979

TECHNICAL WORKSHOP SERIES No. 2

COMPTES RENDUS**
de
L'ATELIER SUR L'ÉCHANGE D'INFORMATIONS D'ORDRE TECHNOLOGIQUE

ACTIVITÉS DE MODÉLISATION RELIÉES À LA RÉDUCTION DES DOMMAGES DUS AUX INONDATIONS

Hull (Québec), du 23 au 25 mai 1979

ÉTUDE N^o 2, SÉRIE DES ATELIERS TECHNIQUES

*These papers reflect the views of the authors and are not necessarily those of the department.

**Ces documents contiennent l'opinion des auteurs qui ne correspond pas nécessairement à celle du Ministère.

FOREWORD

The objective of the workshop was to improve the ability, efficiency, and effectiveness of IWD to fulfil its responsibilities related to modelling activities for flood damage reduction by:

- exchange of experience between regional personnel dealing with practical problems and their solution;
- exchange of technical and research information between IWD research and operational personnel involved in such activities;
- identification of gaps in knowledge requiring research;
- identification of related technical questions requiring further study and/or detailed technical information transfer; and
- identification of improved methods for effecting technological transfer.

Authors were identified to prepare background papers which were distributed well in advance of the workshop to all the participants.

The first day of the workshop (plenary session) briefly addressed three subject areas, each comprised of 3-5 related topics. Each topic was made up of from one to four background papers. Authors presented brief resumés of their papers and, after a group of resumés had been presented, they were opened for discussion for clarification of the subject matter only.

On the second day, the workshop broke into three working groups, each for detailed discussion of one of the three subject areas. All participants had been assigned to a specific working group and were expected to have read the related background papers thoroughly so that they could contribute positively to the discussions. Each author expanded upon his paper with a view to providing more detail where appropriate and highlighting problems and uncertainties. The subsequent discussion after each paper, led by the working group chairman, exposed the problems and gaps in knowledge, and developed recommendations for their solution and/or for subsequent research or management action.

During the concluding plenary session on the third day, working group chairmen presented a report on the discussions and recommendations of their groups. This was then followed by a general discussion which was incorporated as appropriate in the final report by the working group chairmen.

The Proceedings are in two parts: Part I contains summaries (in both French and English) of individual presentations and the ensuing discussion. Part II is a compilation of papers submitted by speakers in the language of presentation only.

AVANT-PROPOS

L'atelier était une tribune pour la communication de connaissances techniques, organisée pour permettre à la Direction générale des eaux intérieures (DGEI) de s'acquitter de meilleure façon, avec plus d'efficacité et d'efficience, de ses responsabilités en matière de modélisation à des fins de réduction des dommages dus aux inondations. Les organisateurs avaient prévu:

- des échanges entre les gens des diverses régions à propos de problèmes particuliers et de leur résolution;
- des échanges de renseignements théoriques et pratiques entre les chercheurs et les autres membres de la DGEI;
- la mise au jour des lacunes de connaissances et des besoins en recherche;
- la détermination des questions techniques connexes qui commandent de nouvelles études ou la diffusion de renseignements techniques détaillés; et
- la mise en lumière de méthodes nouvelles de transfert de l'information.

Des documents de travail, commandés à diverses personnes, avaient été distribués à tous les participants bien avant l'atelier.

Le premier jour de l'atelier (plénière) trois domaines ont été abordés, qui regroupaient de trois à cinq sujets chacun. Les sujets ont fait l'objet d'un à quatre documents de travail, dont les auteurs présentaient un bref résumé. Des échanges sur le domaine suivaient la série d'exposés pertinents.

Le deuxième jour, trois groupes de travail ont été formés, pour étudier un domaine chacun. Les participants, qui devaient avoir lu les documents au complet, ont été appelés à contribuer aux travaux du groupe auquel ils avaient été assignés. Tour à tour, les auteurs ont apporté des précisions sur leurs documents, ajoutant des détails et faisant ressortir les problèmes et difficultés. Ensuite, sous la gouverne du président de groupe, on échangeait sur les lacunes de connaissances et les problèmes, puis on formulait des recommandations axées sur la résolution, la recherche ou la gestion.

Le troisième jour, en plénière, les présidents de groupe ont fait le bilan des échanges et recommandations. Une discussion générale suivit, dont les présidents de groupe rendent compte dans le rapport final.

Le Compte rendu se divise en deux. La première partie réunit les résumés (anglais et français) de toutes les présentations et des échanges qui ont suivi et la deuxième, le texte original des documents présentés.

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PART I

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A. CONCLUSIONS AND RECOMMENDATIONS

1. Future Workshops

In the opinion of the participants and the organizers, the workshop was successful. With careful selection of topic and participants, future workshops could be equally successful.

2. Research Needs

The following four research needs have been assigned the highest priority in terms of modelling for flood damage reduction.

(i) Ice jam flooding.
(ii) Impact of drainage on flooding.
(iii) Frequency analysis of water levels which are autocorrelated and/or dependent on multiple inputs.
(iv) Effect of soil moisture on the precipitation-runoff relation.

3. Operational Needs

The following needs have been classified as urgent for the effective operation of the flood damage reduction program. Assistance should be sought from the private sector, from universities and from outside Canada so that the best possible advice is obtained as soon as possible.

(i) Analysis, prediction and forecasting ice jam flooding.
(ii) Definition of adequate floodproofing.
(iii) Assessment of the benefits of improved forecasts of hydrometeorological inputs for use in real-time flood forecasting.

4. Organizational Needs

The Inland Waters Directorate, preferably with outside assistance, should formulate an overall strategy for research and development in hydrology and hydraulics. In developing this strategy, particular attention should be paid to rectifying existing problems such as:

(i) lack of adequate recognition and use of centres of expertise,
(ii) duplication of effort and lack of co-ordination among research institutes, headquarters units and regional groups,
(iii) poor communication and technology transfer,
(iv) inconsistent quality control of research, and
(v) lack of continuity of headquarters units.

B. WORKING GROUP REPORTS

REPORT ON WORKING GROUP 1

Chairman: T.M. Dick

Open Channel Hydraulic Modelling

An informal approach was adopted for the discussions. Objectives were not sharply defined which therefore demanded a relatively unstructured discussion. Most members in the group participated in an enthusiastic if rather subdued manner. All showed their concern that some substantial outcome should result from our time and effort.

EXCHANGE OF INFORMATION

All but one of the papers was presented in more detail and discussed.

SIMPAK Model J.C.Y. Lee

After further details of the model were presented the following conclusion was reached.

> This model is primarily most useful for planning purposes, especially where reservoirs are contemplated as the proposed method to reduce flood levels. It is also useful for calculating pre-reservoir flows.

It is, however, virtually impossible to establish the error in the calculated output and the model is not directly useful for obtaining flood levels. Plans to link the model to a hydrologic snowmelt model, such as that under development by Quick and Pipe (U.B.C. Watershed Model), are under consideration.

Other Models

Five models were then presented in series and discussed as a group. The models were:

1. HEC-2 - Moose Jaw Application (Steady-State Model)
2. Krishnappan - Unsteady Sediment Transport Model (MOBED)
3. SNC Model - Fraser River Model (Unsteady State).
4. DWOPER - Saint John River Application (Unsteady State)
5. FERNS Model Finite Element - St. Lawrence Application (Unsteady State).

The HEC-2 model is a steady-state model which is adaptable for reaches where details of the flow must be examined. It is not generally suitable for problems and situations where the dynamic models are more suitable. A readily available alternative to HEC-2 is not obtainable.

The SNC, DWOPER and FERNS models are capable of accomplishing much the same thing. FERNS employs a finite element technique and is relatively untried.

The DWOPER model is well documented and versatile and, moreover, regular updating and improvements are available.

The SNC model is fully operational and provides good information. However, as an in-house programme it will eventually suffer from the level of resources and manpower associated with its application. It may also be slightly out of date.

The Krishnappan model, a totally Canadian product, is a brand new model under development for two years in the Hydraulics Research Division at NWRI. It should provide an alternative where sediment transport and bed aggradation and degradation may be an important feature of flooding. As for the SNC, it will require dedicated long-term in-house resources to maintain and develop it in the future.

Only the DWOPER model was shown to have had a field verification run "independent of the calibration process". The Krishnappan model (MOBED) has been verified in the laboratory and work is proceeding on field verification data.

The working group was rather unwilling to make judgements, so the following conclusions are partially the Chairman's.

1. Although there is basically no intrinsic advantage in uniformity, there seems some merit in reducing the unnecessary development of models which basically accomplish the same objectives.

2. Although SNC is satisfactory and FERNS has potential, the DWOPER model seems a better choice and should be given preferential consideration for flood level studies.

3. HEC-2 should be retained and such problems inherent in routines for submerged culverts and discontinuities caused by critical flow should be eliminated. Flow routines for divided flows should be added.

4. It was basically believed that SIMPAK was probably a useful and easy model for planning analysis. Errors in the output remain unknown.

5. MOBED, the Sediment-Krishnappan model, should proceed to check out its performance on full-scale river data. Promotional plans and ongoing resources are also required.

Floods and Ice Problems

None of the models discussed above has any way to deal with ice jams. Ice phenomena and processes were discussed by Dr. G. Tsang and Ms. E. Langley, and a definite requirement is now evident.

Subsequent discussion revealed a total dearth of any coherent plan to establish flood levels where ice was a contributing cause. Corrective action in a research and development sense is urgently required.

COMMUNICATION AND TECHNOLOGY TRANSFER

Barriers to Communication

For an undetermined reason the regional offices develop an unwillingness to discuss problems with one another. Regular management meetings between regions are not designed to promote technical exchanges. Awareness of who is doing what and what is what is virtually non-existent. For example, the initials NWRI were news to many people, and the programmes of NHRI are a mystery. The new structure of the Water Research Advisory Committee may help but that remains to be seen.

However, it should also be mentioned that, in my experience, more effort is required on the listener's part and probably more respect for the integrity and motives of other co-workers in the university, private and public sectors.

The means of communication between provincial agencies and their engineers and scientists are also far from clear and, in addition to the usual problems, poor communication with departmental colleagues in Quebec is aggravated by the language policies, and the sensitivities which have developed.

Proposed Actions to Improve Communication

1. User groups should be organized to meet on a regular basis, to exchange information and developments. This should be on a cost recovery basis.

2. Publications to explain the various models should be prepared and circulated.

3. The recipients of news must listen harder; therefore, they should at least register with the distributor to receive information. The shot gun distribution of publications is inefficient.

4. Centres of expertise should be identified and publicized.

RESEARCH AND DEVELOPMENT

It appears that all regions are intent on research activities to a greater or lesser degree, despite the investment of time and resources in two research institutes. Effective dialogues between researchers and operational personnel are far too rare, and some improvements must be made.

Efficient development of models and the maintenance of expertise require the establishment and recognition of individuals and organizations who know how to develop models. A concomitant condition is the long-term commitment of resources to such a purpose. Resources should include funds for universities and private sector development as well as in-house expertise. All sectors are links in a chain and all require expertise in order to have a useful synergistic effect on real problems.

RESEARCH PRIORITIES

1. The major problem is created by ice. No reliable approach is available and more resources should be applied to study the ice problem. For longer term problems, research studies in ice should be pursued in parallel by DOE researchers, university researchers and also by consultants.

2. The residual problems in HEC-2 should be solved and added to the model.

3. The sediment model of Krishnappan (MOBED) should be brought in line as soon as possible and tested against full-scale data.

POT-POURRI

Although not well defined, there is a general perception that the handling of model applications and research could be better managed. The need to consolidate scarce manpower resources in well-identified groups seems to have general support but is perceived as an arrangement which might detract from regional interests and autonomy.

Recognizing that in-house commentary is either unacceptable to senior managers or is considered biased, it is worthwhile considering appointing a management consultant to investigate and report.

SUMMARY

The choice of models for flood-level prediction is limited. Of primary concern is the reliability of the calculations which are translated eventually into action to reduce flood damage. Computational problems and routines are the main concern of modellers, but management is more concerned with the result. There should be a greater priority given to establishing the reliability and accuracy of models; thus more research and development activity, especially to verify statistically the performance of models, is probably needed.

REPORT ON WORKING GROUP 2

Chairmen: F.I. Morton
W. Nemanishen

Hydrologic Modelling

As Dr. Morton could not be present on May 23, Mr. Nemanishen chaired that session and those of the two following days. At the Working Group session on May 24 the participants were requested to address themselves to the last three of the workshop objectives. These were:

A. Identification of gaps in knowledge requiring research.

B. Identification of related technical questions requiring further study and/or detailed information transfer.

C. Identification of improved methods for effecting technological transfer.

With regard to objectives A and B, the authors listed what they considered to be the most important knowledge gaps and research areas, and the participants rated these on a scale of zero (no importance or regional significance) to three (great importance or regional significance). The resultant priorities are presented in Table I.

Table II provides an expression of opinion on suggested ways to improve technological transfer. The ratings represent the number of participants out of a total of 14 who agreed with the suggestions.

A number of participants filled in a questionnaire on objectives A, B and C. Their suggestions included the need for improved methods of computing runoff from ungauged basins, for studies on forecasting the effects of ice jams on floods, for improved methods of snow modelling, for co-location of WSC gauging stations and AES climate stations, and for better knowledge of what hydrologic modelling is being done at AES and why it is being done. The last suggestion reflects concerns expressed at the joint workshop meeting on May 23.

The information in Tables I and II was presented at the joint workshop session on May 25. There was considerable interest in and discussion of the proposed index of Canadian experts as an inexpensive means for effective technological transfer (Item 3, Table II). The complete lack of interest in a monthly newsletter (Item 11, Table II) was due to the recognition that it would be impossible to justify the establishment of an organization to do the necessary work.

Dr. Morton commented on the lack of scientific input to hydrologic models. Those currently in use have a scientific basis in the law of conservation of mass but the hydrologic cycle is represented by a conglomeration of questionable assumptions given some semblance to reality by fudging of coefficients during calibration. The main problems are in the complexity of the models, in the interdependence of the assumptions and in the acceptance of a model as a package by the user agency. These make it very difficult to detect incorrect assumptions and even more difficult to correct them. As a result, the failure of a model is usually attributed to inadequate input data (see Item 1, Table I) or to inadequate calibration. The most valuable scientific inputs to hydrologic modelling would be:

(1) Research on the hydrologic cycle and the formulation of positive results in such a way that they can be tested with the routine hydrometeorological data that is available for use in hydrologic models. This type of input is implicit in Items 5, 7, 19 and 21 of Table I. The immediate prospects are not good because advances in our knowledge of the complex interactions in the hydrologic cycle are rare.

(2) Critical reviews of the assumptions used in the different hydrologic models. This is relatively easy because the literature is replete with data and theories illustrating the inadequacy of many of the assumptions in particular circumstances. Such reviews would assist user agencies to select the model that is most amenable to correction (i.e. the one with the fewest interdependent assumptions) or to judge whether the poor performance of a model is due to incorrect assumptions, inadequate input data or calibration problems.

(3) Development of methods for objective evaluation of model performance. These would go further than those suggested in Item 15, Table I, and would assist user agencies to compare the merits of different models and to detect the need for improved hydrologic content, better input data or more extensive calibration. The discussions on May 24 and May 25 included reference to a relevant paper entitled "A Criterion of Efficiency for Rainfall-Runoff Models" by M. Garrick, C. Cunnane and J.E. Nash which appeared in the Journal of Hydrology, 36 (1978), 375-381.

Further information that is relevant to the concerns of the Working Group has become available since the end of the Workshop. The first is that the Prairie Provinces Water Board has made a sensitivity analysis of the type recommended in Item 6, Table I, for the Stanford Watershed Model using Davin research basin in Saskatchewan. This analysis is presented in Appendices A and B of the PPWB Streamflow Forecasting Report, Qu'Appelle River to Saskatchewan-Manitoba

Boundary, October 1975. Copies and other information can be obtained from Mr. Nemanishen. The second is that the National Hydrology Research Institute has a project to improve the physical basis and calibration of the Sacramento Watershed Modelling System by adding independent estimates of evapotranspiration from a technique that has been tested over a wide range of environments. Details can be obtained from Dr. Morton.

TABLE I

KNOWLEDGE GAPS AND RESEARCH AREAS IN RUNOFF MODELLING

Item		Priority*
1.	Improved 24- and 48-hour precipitation forecasts.	41
2.	Comprehensive IWD statement re hydrometeorological data required for real-time forecasting.	41
3.	Publication of precipitation and snowcover maps obtained from remote sensing.	40
4.	Appropriate federal agencies perform and publish results of hydrological analyses required for Canadian water resources development.	40
5.	Research on establishing soil moisture-runoff relationships.	39
6.	Water resource modellers provide sensitivity analyses of models for various hydrometeorological input parameters.	38
7.	Develop improved methods of determining antecedent soil moisture indices.	38
8.	Remote sensing of snowpacks.	38
9.	Remote sensing of soil moisture.	37
10.	Develop methods for obtaining integrated areal precipitation.	36
11.	Identify models suited for specific watershed applications.	34
12.	Develop systems for maintaining, updating and distributing selected models under government sponsorhip.	34
13.	Develop improved technology and logistics for acquisition of soil moisture and snowmelt data.	33

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* The highest possible priority is 42.

Item	Priority
14. Water Resources Branch and AES work jointly on all network planning and hydrometeorological projects.	33
15. Standardize methods of objectively comparing models.	31
16. AES provide seasonal temperature and precipitation forecasts.	30
17. AES improve or develop six- to ten-day weather forecasts.	29
18. Further development continue on probabilistic hydrological models.	29
19. Studies be made on all the effects of urbanization on hydrological regimes.	27
20. Accuracy limits be defined for hydrometeorological data required for operational forecasting.	26
21. Identify effects of anticipated climatic change (projected for 1980's to 1990's) on drought-flood regimes in Canada.	23

TABLE II

IMPROVED METHODS FOR EFFECTING TECHNOLOGICAL TRANSFER

Item		Advocates*
1.	Consultative visits.	14
2.	Workshops.	11
3.	Index of Canadian experts (with telephone numbers, etc.).	11
4.	Specific modelling groups with regular meetings.	11
5.	Joint Inland Waters Directorate - Atmospheric Environment Service - Provincial Studies (e.g. Saint John River).	11
6.	Training sessions for specific models.	10
7.	Ottawa library to clip and circulate pertinent abstracts.	9
8.	Improvements to communication channels for upward problem flow.	9
9.	Form coordinating groups for modelling.	8
10.	Improved WATDOC use.	7
11.	Monthly newsletter.	0

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* The highest possible rating is 14.

REPORT ON WORKING GROUP 3

Chairman: R.L. Pentland

Stochastic Analysis and Operational Research

Working Group 3, in summarizing the results of its deliberations, recognized that it had covered only a small fraction of the work underway in Canada related to flood frequency analysis. While using the presented papers as points of initial discussion, it attempted to reach a consensus on several much broader issues than those addressed by the individual papers. Also, in preparing its summary conclusions, the group attempted to relate them as closely as possible to the components of the National Flood Damage Reduction Program.

The Working Group discussed four broad categories: frequency analysis at gauged sites; ungauged sites and regional analysis; large lake/coastal situations and reservoir operation; and physical interventions. For each of these, the Group identified "findings" and "research needs" which attempted to address, respectively, findings of general value to operational people in the Flood Damage Reduction Program, and links which should be made between the program and research programs. The findings and conclusions are summarized in Table 1. In addition, Table 2 provides a summary of more general conclusions reached by the group which are considered important to the FDR program but which could not be classified along with the more specific conclusions. Also included, for general information, is a survey of the numbers of agencies using various frequency analysis techniques (Table 3).

TABLE 1

Summary Conclusions

Category	Findings	Research Needs
Frequency analysis at gauged sites	"Institutionalizing" standard procedures should be avoided (lesson from U.S., Britain, etc.)	There is potential for more exhaustive use of existing data, e.g., more than 1 peak/year.
	There is too much "blind" use of mathematics (sometimes incorrect assumptions about single distribution, stationarity, independence, accuracy of data); too many parameters with too little data.	It is necessary to examine combined probability situations, e.g., confluence of streams, estuarial situations, etc.
	There is an abundance of methods and talent, but lack of criteria to select. Several methods should be tried -- do not underestimate (or overestimate) the value of subjective judgement.	Natural phenomena which introduce hydrologic non-stationarity, e.g., silting, ice jams, should be investigated.
	Recorded streamflow data at extreme values is itself an extrapolation -- maximum use of water level should be made.	Recognition of "memory" in some circumstances, and introduction of techniques to deal with it are required
Ungauged sites and regional analysis	DOE should accept the responsibility and find ways for quality control of analyses in other federal agencies.	Storm/snowmelt frequency may be combined with more physically based models for ungauged sites.
	There is often a tendency towards over-sophistication in models for ungauged sites, i.e., too many "fudge factors".	It is necessary to consider "regions" defined on physiographic/climatic grounds as opposed to a purely geographic basis.
	While the index method is most frequently used for regional analyses, direct regression is gaining favour with many analysts.	Techniques for defining frequencies of durations and volumes should be considered.

TABLE 1 (Contd.)

Category	Findings	Research Needs
Large lake and coastal situations, reservoir operation	There is increasing importance (recognition of) of land use management as opposed to manipulation of water levels.	There is a need for development and verification of better wind set-up/wave run-up models.
	We have been slow to recognize the complexity and the importance of large lake/coastal analyses.	It is necessary to introduce deterministic components, e.g., in tidal areas, and time dependency, e.g., for large lakes.
	Probably the most important aspects of reservoir management models are: need for realism, simplicity, and acceptance by operating agencies.	Better techniques are required to compare non-comparables (economic evaluation, multiple interests which carry vastly different weights, environmental aspects, etc.) in reservoir studies.
Impact of physical interventions on frequency analysis	There is little recognition of the increasingly important physical changes and their impact on flood frequencies (especially drainage).	Much more effort is required on physically based models to examine the effects of drainage, roads, aquatic weeds, forestry practices, etc.
	There has been some research progress in statistical analysis of intervention, but few applications in Canadian water management activities.	There is a need to translate findings from small research basins to larger streams, and more populated regions.

TABLE 2

General Observations

1. Although frequency analysis is the largest source of error in flood risk mapping, it is likely least open to legal problems (do not use inaccuracies in this aspect as an excuse for sloppy mapping and/or hydraulics).

2. Probably 80% of the problem in large lake/coastal situations is due to erosion. It is important to recognize policy differences between flooding and erosion, e.g., related to disaster assistance, effectiveness of physical controls, etc. The need to extend FDR principles to "erosion risk" areas bears looking into, but the advisability from a federal viewpoint is as yet unclear.

3. It is necessary to better define "adequate floodproofing".

4. The time is about right to organize a meeting of all federal and provincial Technical Committee members to exchange experiences, and ensure as much consistency as is deemed desirable in the national program.

TABLE 3

Frequency Analysis Techniques

Preferred Distribution	Number of Agencies Using
Normal	2
Gumbel 1	15
Pearson 3	5
Log Pearson 3	7
Log Normal	9
3-P Log Normal	7
Kritskii-Menkel	1
Number of Agencies	23

OBSERVATIONS OF THE GENERAL CHAIRMAN

E. Watt

1. WORKSHOP ORGANIZATION AND COMPOSITION

1.1 Physical arrangements and logistics

Details concerning location, date, timing, schedule, preparation of preprints, etc. were handled admirably by H.R. Eisenhauer and D.W. Farley. Preprints were available well in advance, the meeting rooms were more than adequate and the schedule appeared to be most appropriate.

1.2 Performance of chairmen and speakers

The three session chairmen carried out their duties in a capable manner. Of particular note was the sincere effort made by all speakers to summarize their papers in five minutes for the opening session. This provided an excellent introduction to the workshop.

1.3 Participants

All regions, as well as the two research institutes and headquarters were represented. In addition there was a limited number of persons from provincial agencies, universities and consulting firms. The overall 'mix' appeared to be appropriate. However, one might have expected a larger representation from NHRI.

1.4 Awareness of FDR program

A number of participants seemed to be surprisingly unaware of the details of the national Flood Damage Reduction Program. This was not to be expected at a workshop on "Modelling Activities Related to Flood Damage Reduction".

2. MODELLING ACTIVITIES

2.1 Assumption

It can be assumed that the papers presented by IWD personnel reflect the IWD modelling activities for flood damage reduction.

2.2 Relevance

The majority of papers dealt with modelling activities that were relevant to flood damage reduction. However, a small number described modelling activities that were not relevant.

2.3 Efficiency and Co-ordination

The modelling activities reported appeared to be largely unco-ordinated, although there were some exceptional examples of co-ordination and co-operation between regions and headquarters. At first glance, such a strategy appears to be inefficient and lead to duplication of effort. This point was noted by the participants in group 1 during a discussion of dynamic open channel models. A similar comment can be made for activities dealing with flood frequency analysis.

2.4 Research and Development vs. Operations

One would have expected regional personnel to be more interested in operational problems, whereas staff from IWD (headquarters), NWRI and NHRI would be expected to be more interested in research and development while keeping in mind the operational problems of the real world.

In fact, only NWRI and IWD headquarters staff reinforced this expectation (there were no papers presented by NHRI staff). Although the majority of papers by regional personnel dealt with operational problems in a practical way, a not insignificant number reported on activities which could be classified as research and development and might have been expected to be carried out by a research institute. Although there are likely very good reasons for such a strategy, it is not an efficient one and, in general, would not be expected to result in high quality research and development.

3. NEEDS

3.1 Technical Needs

The participants were able to list knowledge gaps, research needs, etc. for each of the three groups: (1) open channel hydraulic modelling, (2) hydrologic modelling, (3) stochastic analysis and operational research. These are included in the reports of the group chairmen. These lists should be reviewed and classified according to priority, research effort required, etc. and a plan should be developed to carry out the required research. Worthy of special note is the need for adequate models for analysis of ice jam flooding. This information is urgently required for ongoing mapping programs. Very little assistance can be expected from other countries and the present allocation of funds and manpower to this problem is minimal.

3.2 Communication and Technology Transfer

Groups (1) and (2) discussed communications problems and proposed improved methods for technology transfer. Clearly, this is a major problem which must be solved forthwith. An excellent example of this need occurred during the workshop. One speaker suggested that a model developed by headquarters staff did not apply to his data whereas, in fact, the data had not been classified in the manner required by the model. In this instance, neither the speaker (a 'user') nor the model was at fault. The developers of the model had prepared and distributed a good user's manual but they had since been reassigned so that no one person at headquarters was available to provide instant advice to users.

If technology transfer is not improved, then results of research programs by the research institutes and headquarters units will be of marginal use to regional groups. Moreover, the regional groups need an effective link with research staff so that the latter can be advised of 'real world' problems.

3.3 Organizational Needs

Clearly, in an organization where research and development on one aspect of water resources is carried out by at least nine distinct units [(5 regions, NWRI, NHRI, WRB (Hull), WPMB (Hull)], a great deal of effort must be required to ensure that these activities are appropriate, efficient and effective. Existing problems appear to be:

(i) lack of co-ordination and duplication of effort (2.3),
(ii) inconsistent quality control (2.4),
(iii) lack of identification of centres of expertise,
(iv) poor communication and technology transfer (3.2), and
(v) no obvious overall strategy for research and development which sets out the respective roles of and links among research institutes, universities, headquarters units, private sector, and regions.

4. SUMMARY

In summary, the workshop was well planned and well organized. There was excellent participation in all sessions and those participants who prepared and presented papers should be complimented for their promptness in preparation and for their effective presentation.

The objectives of the workshop have been set out previously. Clearly, all of the objectives have been accomplished. Obviously however, this accomplishment should be regarded as a necessary but not a sufficient condition for effective fulfillment of IWD responsibilities related to modelling activities for flood damage reduction. Research and development are still required to fill gaps in knowledge, particularly in the area of ice jam flooding, and improved methods are needed to communicate the results of this research to operational personnel.

In addition to the stated objectives, an additional outcome of the workshop was the indication of organizational problems which frustrate effective research and development in modelling activities. Although the workshop was restricted to this particular aspect of the broader field of water resources, the same organizational problems may exist in other aspects of water resources research and development.

WORKSHOP EVALUATION

H.R. Eisenhauer

All participants were provided with a workshop critique form and invited to return it to the organizers for comments. Of the 46 participants, 25 returned their critiques. The following is an analysis of these.

1. Facilities

The workshop was held in the Conference Centre of Place Vincent Massey. The first day's proceedings were in the Churchill Room which is designed to hold up to 80 persons in "theatre style". Tables were more or less randomly scattered throughout the room with 3 or 4 chairs placed at each table so that participants could face the front of the room.

For the second day, the workshop broke into three working groups. They were accommodated in the Churchill, Saguenay and Mackenzie rooms. In the two latter rooms, participants sat around a table with the "overflow" accommodated in chairs nearby. The seating capacity around the table in the Saguenay Room was 16, and in the Mackenzie Room it was 12. Participants in the two working groups housed in these rooms numbered 18 and 14 respectively. The plenary session on the third day reconvened in the Churchill Room.

All rooms were provided with a 35-mm slide projector, an overhead projector, a chalk board, and a flip chart.

Critique

Nearly all the respondents were satisfied with the size of the Churchill Room for the plenary sessions, but several commented that the Mackenzie room was too small for the working group session. Most attendees felt the seating arrangements were satisfactory but one thought the arrangement in the Churchill Room was too cluttered while another had difficulty seeing the slides. Everyone thought the equipment was satisfactory.

2. Format

The first day of the workshop (plenary session) briefly addressed three subject areas, each comprised of 3-5 related topics. Each topic was made up of from one to four background papers which had been distributed well in advance of the workshop. Authors were intended to present brief resumés of their papers (not to exceed five minutes) and, after a group of resumés had been presented, they were open for discussion for clarification of the subject matter only. The discussion was also limited to about 5 minutes per paper.

On the second day, the workshop broke into three working groups, each for detailed discussion of one of the three subject areas. All participants had been assigned to a specific working group and were expected to have read the related background papers thoroughly so that they could contribute positively to the discussions. Each author was allotted about 10-15 minutes to expand upon his paper with a view to providing more detail where appropriate and highlighting problems and uncertainties. The subsequent discussion after each paper, led by the working group chairman, was to expose the problems and gaps in knowledge, and to develop recommendations for their solution and/or for subsequent research or management action.

During the concluding plenary session on the third day, working group chairmen were to present a report on the discussions and recommendations of their groups, which would then be followed by a general discussion. Each report and discussion was allotted about 45 minutes.

Coffee breaks of about 20 minutes were scheduled in the morning and afternoon of each day. In addition, an informal reception (pay-bar) was held following the first day's proceedings in an attempt to "break the ice" and promote informality.

Critique

Ninety percent of those replying said that the number of topics covered in the workshop was about right, although two people thought there were too many. One participant felt that the presentations were too informal but everyone else responded that the degree of formality was about right. Nearly 20% of the respondents thought the time allowed for presentation of the papers was too short, and about ten percent had the same comment in regard to the length of the discussion periods. Four of those responding thought that discussion should be restricted to the end of each half-day and a similar number felt that the discussions were not interactive or spontaneous. One person suggested that this was because the group was too large and diverse, and another thought it was because too many of the authors were trying to "sell their work" rather than expose the problems and difficulties they encountered. Most of the participants agreed that the Chairmen had done a good job, some suggesting that it might be improved by meeting with authors before the working group sessions to identify controversial issues, by having the papers reviewed beforehand by experts to identify a focus for discussion, and by ensuring that all papers received equitable treatment in the time available for presentation and discussion. One respondent complained that his working group chairman interjected his own views too frequently into the discussion. The social arrangements appeared to be generally satisfactory and of value in providing an opportunity for one-to-one discussion, although the reception might have been longer.

3. Material

All active participants (authors and chairmen) were sent a detailed description of the workshop about 10 weeks in advance which included the objective of the workshop, its format and agenda, and the responsibilities of the participants. All authors were asked to prepare background papers (limited to 10 pages each) and brief resumés. Most of these were received in good time and forwarded to all attendees about a month before the workshop. Each attendee was asked especially to read those papers which the working group, to which he was assigned, would be discussing in detail.

Critique

Although most of the respondents agreed that the purpose of the workshop was clear and had been achieved, two did not. Nearly 90% agreed that the workshop covered the subject and all but one respondent felt that the topics were generally pertinent to the workshop subject. Some respondents thought that the papers could have been directed more specifically to flood damage reduction than they were and that selection could have been improved by a paper review group. Ninety to 100% of the respondents thought that the preliminary material (background papers) was adequate, covered the topics well, and was of use to them. Two people remarked that many authors did not follow the instructions regarding the length of the papers and the specific identification of problems, issues, gaps in knowledge, etc. which would have served to promote useful discussion. Nearly 90% of the respondents felt that the quality of the presentations was adequate. One attendee remarked that most of the authors tended to "rehash" their papers for the second day's presentation, while another did not like the presentations being split into a general one the first day and a more specific one the second day. One in seven of the respondents felt there were too few visual aids used in the presentations and one attendee recommended that a supply of transparencies and pens should have been made available to prepare summaries on overhead projectors. All respondents agreed that the publication of Proceedings would be an adequate follow-up to the workshop. Three people stressed that the recommendations arising from the working groups (as recorded by the working group chairmen) should be referred to IWD senior management for action.

4. Participants

The subject matter of the workshop, together with specific papers and authors, was recommended by an *ad hoc* organizing committee to the IWD Directors' Committee for approval. This established the active participants at the workshop. In addition, Regional and Headquarters Directors nominated additional attendees for final approval by the Directors' Committee. There were 46 participants of whom 30 played an active role as an author or chairman (see List of Participants). Of the attendees, 33 were from IWD (17 Regional, 11 Headquarters, 5 from the Research Institutes), two from other federal agencies, six from provincial agencies, four from universities, and one from the private sector.

Critique

One in ten of the respondents felt there were too many participants, and an equal number thought there were too few. Twenty percent of the respondents thought that the selection of participants could be improved by increasing the participation of identified experts (consultants and provincial) who are actually doing work in the field.

5. General

All respondents felt that the workshop was a useful means to transfer technology. One person thought the in-depth working group session was the most useful and that the plenary sessions were not of particular value. Another recommended that short courses put on by experts on specific topics would be of value.

The workshop was personally useful to all the respondents. Two people noted that the workshop was of particular value because of the contacts they were able to make with others working in the field.

One third of the respondents commented very favourably on the quality of the organization, planning and running of the workshop.

6. Recommendations

Eighty-five percent of the respondents made recommendations which they thought could improve future workshops. These are summarized below. In most cases, the recommendations originate from an individual attendee. Where more than one participant made a recommendation, the number is indicated in parentheses.

6.1 Facilities

- the room size should be such that all participants can view the slides and other visual aids without difficulty
- a projectionist should be available
- transparencies and pens should be available for overhead projections
- seating around small tables is an excellent idea

6.2 Format

- more time should be allotted for presentation and discussion of the papers (5). This could be accomplished by narrowing the subject area and selecting fewer topics (4)
- discussion should follow each paper (2) with an overview provided by the chairman at the end of the day
- chairmen should ensure equitable time distribution for all presentations and discussions
- chairmen should meet with authors beforehand to identify controversial issues
- the session chairmen should meet to plan a common approach to the working group sessions and their responsibilities
- a biographical sketch of the authors should be provided with the background papers or in their introduction
- session chairmen should entertain only questions or discussion that pertains to the objective of the workshop
- the workshop should concentrate on the detailed presentations and discussion (4) and have the working group chairmen provide a summary overview of all the papers in the first day (1-2 hrs). (Note - in contrast, one attendee commented that the "first day was most informative to the audience", and a second that "the third day was very successful")
- the social get-together should be held the last night of the workshop in some hotel - the one-hour open bar period was too brief

6.3 Material

- all authors should follow the instructions regarding the length of the background papers (10 pages) and their emphasis (identification of problems, knowledge gaps, how things could have been done better, etc.) (6)
- papers should be solicited, critically reviewed by experts, revised in accordance with guidelines, and selected on the basis of relevance of the workshop topic (5)
- discussion papers should be produced for the workshop in response to the background papers to expose all critical views and promote lively and productive discussion (2)

- all speakers should be encouraged to use visual aids
- recommendations as prepared by the working group chairmen should be highlighted in the Proceedings and not summarized or otherwise "watered down"
- workshop recommendations should be referred directly to senior management for consideration, action, and implementation (3)
- in the oral presentations, the authors should strive to present the technical material in a concise manner

6.4 Participants

- care should be exercised to ensure that all participants are technically competent and will contribute positively, especially including those who have actually done the work in the subject area (4)
- every effort should be made to include experts from provincial governments and the private sector (4)
- there should be fewer participants in each working group
- there should be more participants in each working group

6.5 General

- this type of workshop should be held more frequently
- short courses put on by experts in specific subjects should be considered
- "user group meetings" should be considered
- small follow-up workshops should be organized to deal with specific topics such as one-dimensional hydrodynamic modelling, intervention analysis, watershed forecasting
- workshop critiques should be collected at the end of the workshop

C. ABSTRACTS OF BACKGROUND PAPERS

SIMPAK for River Basin Studies

J.C.Y. Lee and R.O. Lyons

In several studies in the Pacific and Yukon Region, streamflow routing models have been used to determine the downstream effects of various reservoir or diversion projects. In the earlier studies, the SSARR program was adopted as one of the vehicles for routing studies in the region. However, only a fraction of the capabilities of SSARR were required for this task. The desired result could be achieved by simply using the routing methods of the SSARR program and building a separate program framework to do all the data handling. Thus, the simulation package (SIMPAK) for river modelling was developed and used by the Vancouver office of Inland Waters Directorate for several river basin studies. It has been used to perform river routing and reservoir regulation, critical flood flow forecasting and other operations involving manipulation of streamflow data. The critical flood flow forecasting was developed specifically for day-to-day reservoir flood control operation. The technique derived for the forecasts is a simplified approach yet provides a workable and useful tool for reservoir operation. This paper describes the features of the program, provides a comparison with the SSARR model, and describes some applications.

Development of a Real-Time Operational Model for the Ottawa River - A Progress Report

G.A. Nix

In May, 1974, a Canada-Quebec agreement was signed that initiated studies on flooding and low water problems in the Montreal Region. The study report recognized that any potential for reducing flooding in the Montreal Region lay in optimizing the operation of the many reservoirs in the Ottawa River basin. As a result of the recommendations in the report, the Ottawa River Regulation Planning Committee was formed in February, 1977. The Committee formed a Subcommittee to recommend criteria for regulation of the Ottawa River, taking into account hydro-power production, flood protection, navigation, low water problems, water quality needs and recreation. A Working Group was then formed to develop a mathematical model for the regulation of the Ottawa River. This report describes the progress towards the development of the model.

HEC-2 Application at Moose Jaw - Input Requirements and Model Performance

R.D. Hofer

On March 8, 1978, IWD, Western and Northern Region, entered into an agreement with the Saskatchewan Department of the Environment to conduct various hydraulic investigations under the Canada-Saskatchewan Flood Damage Reduction Agreement. The initial contract called for the development of 1:100 and 1:500 floodlines along the Moose Jaw River, Thunder Creek and Spring Creek. A floodway along the Moose Jaw River was also called for. The HEC-2 model was acquired for this project to carry out the streamflow modelling. This paper describes the procedures used, the modelling problems, and the results.

Mathematical Modelling of the Fraser River System

M. Sydor and P. Zrymiak

A one-dimensional finite-difference hydrodynamic model has been applied to the complex channel system of the lower Fraser River. The input data was a set of approximately 700 cross sections collected by DPW in 1962 and 1976. Initial success using the model was realized by calibrating to a discharge of 350,900 cfs discharge on July 12, 1976, and by simulating the June 4, 1976 river condition when 227,500 cfs was recorded. With such encouraging results, the studies are continuing and various periods are to be studied in order to determine the range of accuracy possible in using this model.

Application of the DWOPER Model to the Lower Saint John River

H.S. Belore, A.K. Bhattacharyya, and C.T. Bishop

An unsteady flood routing model, referred to as DWOPER (an acronym for Dynamic Wave Operational Model), has been applied along the lower Saint John River. The model, which is representative of the state of the art in unsteady flow modelling, was selected for this application after considering a number of available hydraulic routing techniques. The model accounts for factors such as the downstream effects of the Reversing Falls and flow over the Trans Canada Highway to large storage areas in the Grand Lake area. Model calibration and verification simulations resulted in a good reproduction of historical flood levels. Subsequently, flood profiles associated with the 1:20 and 1:100 year recurrence interval flood events were determined along the lower Saint John River from McKinley Ferry to Lower Jemseg.

Application of Finite Element Method in River Network Hydraulics

S.M.A. Moin

This paper describes a finite element model of the unsteady flow regime in open-channel networks. Mathematically, one-dimensional unsteady flow hydraulics of open channels is described by a pair of nonlinear, partial differential equations of hyperbolic type and is solved by adopting linear elements.

A Predictor-Corrector method for solving the nonlinear equations is utilized. Simulation capabilities of the computerized model were investigated in determining the steady-state profile for the St. Lawrence River. Limited scale sensitivity analysis was conducted to study the influence of the variation of the weighting parameter.

A comparison of the model output with the observed events and/or calculated values indicates the suitability of the model from the viewpoints of simulation capabilities, adoption of higher time steps, conservation of mass, flexibility of operation, and handling of complex network configurations.

Mathematical Modelling of Unsteady Flows in Mobile Boundary Channels

B.G. Krishnappan

A numerical model is described which solves the unsteady flow equations for a mobile boundary channel. This model is capable of predicting the flood-wave characteristics as well as the environmental impacts of man's activities in river basins. The model was verified under laboratory conditions, and a reasonably good agreement between the model predictions and laboratory measurements was observed.

Ice Jamming Concerns and Operational Solutions

E.R. Langley

Examination of ice jamming case histories and the limited technical information available on the subject indicate that there is a need to develop guidelines for including the effects of ice jams in flood risk mapping work and an even greater need to obtain reliable information on ice jam events. In the meantime, it is proposed that each location be considered separately to determine the best method of including the ice jamming aspects in flood risk mapping.

Hydraulic Aspects of Ice Jamming

G. Tsang

The physical factors affecting ice formation and ice jamming are reviewed. Although it is possible to predict the occurrence of ice jamming with a certain degree of confidence, many problems remain unsolved including ice jam flooding characteristics, instability of ice cover, and the evolution of dynamic jams and their resistance to flow.

Meteorological Inputs for Hydrologic Models

R.G. Lawford

Precipitation and temperature are the principal controls of the hydrologic cycle. Historical and real-time data are available from AES, and forecasts of precipitation and temperature are available from Regional Weather Centres and from the Canadian Meteorological Centre. New technologies and their application to hydrologic models are discussed and a brief outline of changes that could be made to the models are suggested to provide more realistic representations of the meteorological processes. Recommendations are made for further improvement of hydrological models and their application to a flood-damage warning system.

The Need for a Regional Unit Hydrograph Analysis of Ontario

D. Kimmett

Many of the credibility problems associated with past hydrologic studies are related to the fact that each individual has a different understanding of unit hydrograph theory and how it is best applied. It is proposed that Water Survey of Canada develop the necessary in-house expertise and publish unit hydrograph(s) on a continuing as required basis for appropriate hydrometric stations. Recent technological advancements, the need for standards in hydrology generally, and the availability of sufficient data are further incentives for initiating such a program at this time. An informal canvass of several data users drew favourable responses to the concept. A conceptual look at what should be published is provided.

Runoff from Developing Areas

J. Marsalek

Runoff from developing areas is of particular interest in connection with flood modelling because the land development dramatically alters the watershed hydrological response. During the past 15 years, a number of models for modelling of runoff from developing areas have been developed and most of the models are now fully operational. Further development and refinement of these models is currently underway with particular reference to the assessment of the effects of urbanization on the watershed hydrological regimen, modelling of infiltration, and modelling of flood damages.

Flood Forecasting in the Saint John River Basin - Mathematical Modelling Activities

D.A. Smith

The development of a flood forecasting system for the portion of the Saint John River Basin in the Province of New Brunswick is reviewed. The paper progresses, in chronological order, from the time when there was no forecasting system, through the first efforts to calibrate the SSARR model for the basin and the forecasting operations during the 1973 record historic flood to the present. Emphasis is placed on the mathematical modelling activities and on recent efforts to improve the forecasting system being carried out under the Canada- New Brunswick Flood Damage Reduction Program. The major concerns of the Technical Sub-Committee for Flood Forecasting are presented, and the activities presently underway, or planned, to overcome some of these difficulties are discussed.

Data Network Design for Watershed Forecasting

D.C. Ambler

The paper describes the data network required to support a flood forecasting operation with particular reference to the Saint John River. Sources of error in stream gauge measurements and their influence on the hydrologic model output and forecasting are discussed.

Ottawa River - Comparison of the Hydro Québec and SSARR Models

G.E. Brown

An analysis of the forecasted watershed and streamflow discharge results for five Ottawa River Basin watersheds was undertaken to compare the Hydro Québec and SSARR natural inflow models. The Hydro Québec model generally out-performed the SSARR model for these five particular watersheds. With respect to 10-day forecast results there was a noticeable deterioration in the SSARR model results, whereas the Hydro Québec model showed a marginal deterioration from each day to the next of the forecast period. It is pointed out that the Hydro Québec model was designed for relatively large watersheds calibrated to moving 7-day mean daily inflows for reservoirs at the extremity of each basin. Thus, performance of the Hydro Québec model is uncertain for hydrometrically gauged watersheds, since the five watersheds in question were calibrated to computed reservoir inflow data.

Short-Term and Long-Term Hydrometeorological Epochs

W. Nemanishen

Scientific evidence is becoming stronger to indicate that extraterrestrial factors, such as solar activity and lunar cycles, influence atmospheric and oceanic circulation which then combine to produce prolonged droughts or floods in specific locations of the Northern Hemisphere. The studies of several prominent world scientists are highlighted to show how their findings can be used to improve IWD studies of western Canadian flood problems. A case history is presented to illustrate the difficulty of assigning correct return periods for rare flood events and the valuable contribution that runoff modelling can make in these cases.

The Three Parameter Lognormal Distribution Applied to Regional Flood Frequency Analysis by the Index Flood Method

R. Condie

Regional flood frequency analysis by the Index Flood Method is in fairly common use when estimates of flood flows of specific probabilities are required at ungauged river sites. Use of this method usually assumes that the Gumbel 1 distribution gives a fair representation of the flood frequency régime throughout a homogeneous region, and the most commonly used homogeneity test is based on sampling from that distribution. Some reservations have arisen regarding the adequacy of the Gumbel 1 distribution, but use of other distributions may have been inhibited by lack of appropriate homogeneity tests. The three parameter lognormal distribution is generally superior to the Gumbel 1 in Canada. Reasons for this choice are given and the derivation of a homogeneity test is described. Advantages and disadvantages of the Index Flood Method are also discussed in relation to the method of direction regression on the T-year flood.

Flood Frequencies by the Kinematic Wave Model

R.O. Lyons

The Kinematic Wave model was used by Inland Waters Directorate, Pacific and Yukon Region, as one of several methods to estimate design floods for the Alaska Highway Pipeline investigation. This paper briefly describes the basic equations forming the model and discusses the way the model was calibrated for the southern Yukon. The model gives a convenient method for directly estimating flood frequencies at ungauged basins. A fairly large effort in analysis of meteorological data is required, but once this is done, frequency curves can be developed quickly, cheaply, and directly.

Flood Frequency Analysis by Power Transformation

R.S. McAully

This paper describes a technique for flood frequency analysis by fitting the normal probability distribution to an annual flood series after applying what has been called a "power transformation". The computer program to implement this power transformation is described and some results from analyzing 35 streams in British Columbia are presented. The power transformation method, which is very easy to apply, generally gave excellent agreement with the established log-Pearson III method.

Flood Frequency Analysis Concerns in the Atlantic Region

J.D. Keefe

This paper highlights some of the technical problems being experienced in the Atlantic Region in implementing the flood risk mapping program, the approaches to overcome these problems, and the areas in which more applied research and development and/or further guidelines are required. The discussion is supported by examples from various existing or planned investigations where available.

Design Flows for the Shakwak Project

R. Kallio

Design flows for the Shakwak Highway Project and the reconstruction of the Haines Road-Alaska Highway have been estimated by a number of methods. The application of the various methods is outlined and some representative results given.

A Flexible Family of Stochastic Models

K.W. Hipel

The family of Box-Jenkins models, and other closely related stochastic models, form a group which can be used to help solve many types of problems which arise in hydrology. A model which is of particular importance in hydrology is the intervention model. This model can be employed to determine the effects of man-induced or natural interventions upon the mean level of a time series, to fill in missing observations in a time series, and to answer other kinds of similar problems. Following a description of the intervention model, it is explained how the most appropriate type of intervention model can be selected to fit to a specific data set by adhering to the identification, estimation, and diagnostic check stages of model development. A number of examples confirm the importance of this model for modelling Canadian hydrological data.

Hydrologic Extreme Value Model and Risk Analysis Applied to Flood Plain Management

F. Ashkar, N. El-Jabi, and J. Rousselle

Rational planning of flood plains may be undertaken by separating them into different zones to control their development, and basing insurance premiums on flood damage risk for each zone. The risk may be established from the relationship between the return period, or the occurrence of a given flood, and the flood level. A mathematical model based on the extreme values theory in stochastic processes has been established where the distribution function of these maximum exceedances in an interval of time (0,t) could be found. Bayesian and maximum likelihood approaches are followed in order to find the distribution function of the exceedance for a given return period when uncertainty arises in the parameters of the distribution function describing the model. The risks encountered in evaluating this exceedance using different lengths of period of record are compared.

Hydro Configuration Modelling System

D.W. Farley

A generalized modelling package referred to as HCMS (Hydro Configuration Modelling system) has been developed for studying multi-reservoir river systems. The package is particularly applicable to basins in which there is an existing hydroelectric generating capacity and/or a potential for further development. The package is used to evaluate proposals relating to: increased reservoir storage devoted to flood protection; new reservoirs; new installed turbine capacity; environmental requirements in terms of constrained reservoir levels or minimum flows; diversions of flow; and increased channel discharge capacity. By comparing the results of different simulated operation policies and configurations with each other and with historical performance, the planning team can readily assess the impact on flood protection, energy production, and environmental concerns. In order to expedite the modelling phase of studies, the package is endowed with a realistic optimal seeking solution routine. This optimal seeking solution routine employs a linear programming sub-routine available from the IBM package SL-MATH. As opposed to other applications of linear programming to water resource systems studies, the optimal seeking solution routine of this model does not assume perfect hindsight of time series data, nor is it restricted to critical periods of record to conform with dimensionality requirements common to other optimization models.

A Procedure for Determining Flood Frequencies for Great Lakes Shoreline

D.W. Brown, R.J. Moulton, and S.M.A. Moin

Procedures are presented which can be used to determine flood frequencies on the Great Lakes. The procedure varies according to the availability of data. The procedure is relatively simple near gauges where there are records of peak flood levels. If such data is not available, the procedure becomes more complex, as the flood frequencies must be derived by calculating wind set-up and run-up distributions from climatalogic data and combining these frequency distributions with mean water level frequency distribution for the lake.

PREMIÈRE PARTIE

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Flood frequencies by the kinematic wave model	R.O. Lyons	85, 354
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A flexible family of stochastic models	K.W. Hipel	86, 422
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3-2 Techniques de recherche opérationnelle

Hydro configuration modelling system	D.W. Farley	87, 447

3-3 Les côtes de lac

A procedure for determining flood frequencies for Great Lakes shoreline	D.W. Brown R.J. Moulton S.M.A. Moin	87, 456

A. CONCLUSIONS ET RECOMMANDATIONS

1. Prochains ateliers

Les participants et les organisateurs considèrent l'atelier comme une réussite. Si les sujets et les participants sont bien choisis, les prochains ateliers pourraient donner d'aussi bons résultats.

2. Besoins en recherches

À la lumière des échanges, quatre points devront faire particulièrement l'objet de recherches et de modélisation dans l'optique de la réduction des dommages dus aux inondations:

(i) les inondations imputables aux embâcles;
(ii) l'effet du drainage sur les crues;
(iii) l'analyse de fréquence relative aux niveaux d'eau où il y a autocorrélation ou jeu de facteurs multiples, ou les deux; et
(iv) l'effet du taux d'humidité du sol sur la relation précipitation/écoulement.

3. Besoins opérationnels

Pour que le programme de réduction des dommages dus aux inondations produise les effets escomptés, il faut, sans tarder, chercher à obtenir l'aide du secteur privé, des universités et de l'étranger; il s'agit d'obtenir au plus tôt les meilleurs conseils et avis, à trois égards:

(i) l'analyse, la prédiction et la prévision des embâcles;
(ii) l'établissement de normes de protection contre les crues; et
(iii) l'évaluation des avantages inhérents à de meilleures prévisions de facteurs hydrométéorologiques servant à la prévision des crues en temps réel.

4. Besoins relatifs à l'organisation

La Direction générale des eaux intérieures devrait, de préférence avec l'aide de ressources de l'extérieur, formuler une stratégie globale pour la recherche et le développement en hydrologie et en hydraulique. Lors de l'établissement de la stratégie, il y aurait lieu de tenir grand compte de certaines lacunes à combler, comme:

(i) la reconnaissance et l'utilisation insuffisantes des autorités compétentes;

(ii) le double emploi et le manque de coordination entre les instituts de recherche, les services de l'administration centrale et les éléments régionaux;
(iii) les failles de communication et de transfert de connaissances techniques;
(iv) l'inconsistance du contrôle exercé sur les recherches; et
(v) l'instabilité relative des éléments de l'administration centrale.

B. RAPPORTS DES GROUPES DE TRAVAIL

RAPPORT DU GROUPE DE TRAVAIL 1

Président: T.M. Dick

Modélisation hydraulique des canaux ouverts

Les discussions se sont déroulées en table ronde, sans structure précise, puisque les objectifs n'avaient pas été nettement définis. La participation de la plupart des membres du groupe a été enthousiaste bien que plutôt modeste. Tous les membres ont exprimé l'avis que les efforts du groupe devaient donner des résultats appréciables.

ÉCHANGE D'INFORMATION

Tous les documents sauf un ont été présentés en détail et mis en délibération.

Modèle SIMPAK J.C.Y. Lee

Une fois présentés des détails sur le modèle, on a tiré la conclusion suivante:

> Ce modèle est le plus utile à des fins de planification, particulièrement dans les cas où des réservoirs sont proposés pour écrêter les crues. Il sert aussi au calcul des débits avant l'aménagement de réservoirs.

Cependant, il est pratiquement impossible d'établir l'erreur de rendement calculé, et le modèle n'est pas directement utile pour déterminer les niveaux des crues. On est en train d'étudier la possibilité de le jumeler à un modèle hydrologique de fonte des neiges tel que celui qu'élaborent actuellement Quick et Pipe (modèle de bassin hydrographique de l'Université de la Colombie-Britannique).

Autres modèles

Cinq modèles ont été présentés consécutivement et traités collectivement:

1. HEC-2 - Application à Moose Jaw (modèle à état stable)
2. Krishnappan - Modèle à état instable de transport de sédiments (MOBED)
3. Modèle SNC - Modèle du fleuve Fraser (état instable)

4. DWOPER - Application à la rivière Saint-Jean (état instable)

5. Modèle FERNS à élément fini - Application au Saint-Laurent (état instable).

Le modèle HEC-2 est un modèle à état stable qui peut être adapté aux étendues dont les débits doivent être examinés en détail. Il ne convient pas, en général, aux problèmes et aux situations que prévoient bien les modèles dynamiques. On ne peut obtenir facilement une variation du modèle HEC-2.

Les modèles SNC, DWOPER et FERNS donnent à peu près les mêmes résultats. Le FERNS fait intervenir une technique à élément fini et n'a presque pas été éprouvé jusqu'à maintenant.

Le modèle DWOPER est souple et fermement appuyé sur des documents, et peut être mis à jour et amélioré périodiquement.

Le modèle SNC est entièrement opérationnel et fournit des renseignements valables. Toutefois, les ressources financières et humaines affectées à son application en tant que programme interne deviendront insuffisantes avec le temps. De plus, il se peut qu'il soit légèrement dépassé.

Le modèle Krishnappan, produit entièrement canadien, est un modèle tout à fait nouveau qu'élabore depuis deux ans la Division de la recherche hydraulique de l'INRE. Il devrait constituer une solution de rechange dans les cas où le transport de sédiments ainsi que l'alluvionnement et l'affouillement peuvent être des facteurs d'inondation importants. Pour ce qui est du modèle SNC, il devra être entretenu et développé grâce à des ressources et à des efforts internes à long terme.

Seul le modèle DWOPER a été vérifié sur le terrain indépendamment du procédé d'étalonnage. Le modèle Krishnappan (MOBED) a été vérifié en laboratoire, et sa vérification sur le terrain se poursuit.

Le groupe de travail ayant hésité à porter des jugements, les conclusions suivantes sont en grande partie celles de son président.

1. Bien que l'uniformité n'ait aucun avantage intrinsèque, il semble souhaitable de réduire l'établissement de modèles qui donnent fondamentalement les mêmes résultats.

2. Bien que le modèle SNC soit satisfaisant et que le FERNS soit prometteur, le modèle DWOPER semble être meilleur et devrait faire l'objet d'une évaluation préférentielle aux fins des études sur les niveaux des crues.

3. Le modèle HEC-2 est à retenir en éliminant les problèmes inhérents au programme applicable aux passages d'eau submergés et aux discontinuités causées par des écoulements critiques. Il faudrait y ajouter des programmes visant les écoulements divisés.

4. On estime, en principe, que le SIMPAK est probablement un modèle utile et facile à employer pour l'analyse de planification. Cependant, les erreurs de rendement demeurent inconnues.

5. Il faudrait poursuivre la vérification du rendement du modèle MOBED, modèle de sédimentation de Krishnappan, avec des données hydrographiques véritables de pleine envergure. Des plans promotionnels et des ressources permanentes s'imposent.

Inondations et problèmes posés par les glaces

Aucun des modèles susmentionnés ne peut tenir compte des embâcles de glace. Les phénomènes et processus relatifs aux glaces ont été traités par M. G. Tsang et Mme E. Langley, et un besoin marqué a été mis en évidence.

Les discussions subséquentes ont révélé l'absence de tout plan cohérent de détermination des niveaux des crues causées en partie par les glaces. Il faut de toute urgence combler cette lacune par la recherche et le développement.

COMMUNICATIONS ET TRANSFERT DE CONNAISSANCES TECHNIQUES

Obstacles aux communications

Pour une raison indéterminée, les bureaux régionaux sont de moins en moins disposés à discuter de problèmes entre eux. Les réunions régulières des cadres de différentes régions ne sont pas organisées de manière à favoriser les échanges de connaissances techniques. La connaissance des activités menées par les autres et de leurs responsables est pratiquement nulle. Par exemple, de nombreuses personnes ne connaissaient pas jusqu'à présent le sigle anglais NWRI ni la teneur des programmes de l'INRH. La nouvelle structure du Comité consultatif de la recherche sur les eaux aidera peut être à débrouiller l'écheveau, mais il reste à en faire la preuve.

Je dois souligner que, selon mon expérience, le véritable besoin est celui d'être à l'écoute des autres et probablement de respecter davantage l'intégrité et les buts de ses collègues dans les secteurs universitaire, privé et public.

Le moyen de communication entre les organismes provinciaux et leurs ingénieurs et scientifiques est loin d'être clair et, en plus des difficultés habituelles, la qualité des communications avec les autorités du Ministère au Québec sont aggravées par la politique linguistique et les susceptibilités du personnel de cette région.

Mesures proposées pour améliorer les communications

1. Il faudrait organiser des groupes d'utilisateurs qui se réuniraient régulièrement pour échanger des renseignements et des connaissances, des dispositions étant prises pour recouvrer les dépenses soldées à cette fin.

2. Des publications expliquant les différents modèles devraient être rédigées et distribuées.

3. Puisque ceux qui reçoivent les nouvelles doivent être plus attentifs, ils devraient au moins s'inscrire aux listes des distributeurs pour recevoir des renseignements. La distribution des publications à l'aveuglette est inefficace.

4. Il faudrait relever et faire connaître les centres de connaissances techniques.

RECHERCHE ET DÉVELOPPEMENT

Il semble que toutes les Régions soient résolues à mener des activités de recherche, sur une échelle plus ou moins grande, malgré le temps et les ressources consacrées à la recherche par deux instituts. Les dialogues efficaces entre les chercheurs et le personnel d'exploitation sont beaucoup trop rares; des améliorations s'imposent.

La mise au point efficace de modèles et le maintien des connaissances techniques nécessitent la désignation et la reconnaissance de personnes et d'organismes sachant comment établir des modèles, et l'engagement à long terme de ressources à cette fin. Ces ressources doivent comprendre des fonds destinés à augmenter les connaissances des universités et du secteur privé ainsi que celles du Ministère. Tous les secteurs sont des mailles d'une même chaîne et ont besoin de connaissances techniques afin d'avoir un effet de synergie utile sur les problèmes pratiques.

IMPÉRATIFS DE RECHERCHE

1. Le problème majeur est causé par les glaces. Aucune ligne de conduite fiable n'a été tracée à ce sujet; il faut augmenter les ressources consacrées à l'étude du problème des glaces. Dans le cas des difficultés à long terme, les recherches sur les glaces doivent être réalisées de concert par les chercheurs du MDE, des universités et du secteur privé.

2. Il faut combler les dernières lacunes du modèle HEC-2.

3. Le modèle de sédimentation de Krishnappan (MOBED) doit être mis à l'essai le plus tôt possible avec des données d'envergure normale.

QUESTIONS DIVERSES

Bien qu'elle soit diffuse, l'impression générale est que l'application des modèles et la recherche peuvent être mieux administrées. Le besoin de concentrer les maigres ressources humaines dans des groupes bien délimités semble être reconnu par tous mais est considéré comme un arrangement pouvant détourner l'attention des intérêts des Régions et réduire l'autonomie régionale.

Comme les observations internes sont soit inacceptables pour les cadres supérieurs, soit jugées déformées, il y a lieu de songer à charger un conseiller en gestion de mener une enquête et d'en rendre compte.

SOMMAIRE

Le choix des modèles de prévision des niveaux des crues est restreint. Une des considérations principales est la fiabilité des calculs devant se traduire par des mesures de réduction des dommages d'inondation. Les problèmes et les programmes de calcul sont les préoccupations principales des modélisateurs, mais les gestionnaires s'intéressent surtout aux résultats. Il faudrait mettre davantage l'accent sur la détermination de la fiabilité et de la précision des modèles, ce qui appelle l'augmentation des activités de recherche et de développement, surtout pour vérifier statistiquement le rendement des modèles.

RAPPORT DU GROUPE DE TRAVAIL 2

Co-présidents: F.I. Morton
W. Nemanishen

Modélisation hydrologique

Puisque M. Morton ne pouvait pas être présent le 23 mai, M. Nemanishen a présidé la séance ainsi que celles des jours suivants. Au cours de la séance tenue par le groupe de travail le 24 mai, on a demandé aux participants d'examiner les trois derniers objectifs de l'atelier, soit:

A. Détermination des lacunes des connaissances qu'il faut combler par la recherche.

B. Détermination des questions techniques connexes devant faire l'objet de nouvelles études ou de transferts de renseignements détaillés.

C. Détermination des méthodes améliorées de transfert de connaissances techniques.

Pour ce qui est des objectifs A et B, les auteurs ont énuméré les lacunes des connaissances et les domaines de recherche qu'ils estimaient les plus importants, et les participants leur ont attribué des cotes de zéro (aucune importance ou envergure régionale) à trois (grande importance ou envergure nationale). L'ordre de priorité qui est ressorti de cet exercice figure au tableau I.

Le tableau II présente des recommandations personnelles sur les moyens d'améliorer le transfert de connaissances techniques. Les cotes représentent le nombre de participants, sur un total de 14, qui sont en faveur des recommandations.

Bon nombre de participants ont rempli un questionnaire sur les objectifs A, B et C. Parmi leurs suggestions figuraient l'amélioration des méthodes de calcul du ruissellement depuis des bassins non contrôlés, des études de prévision des effets d'embâcles de glace sur les crues, l'amélioration des méthodes de modélisation nivale, le fusionnement des stations hydrométriques de la DRHC et des stations climatologiques du SEA, et l'amélioration de la diffusion d'information sur les travaux de modélisation hydrologique du SEA et leurs buts. Cette dernière recommandation traduit des considérations exprimées à l'occasion de la séance plénière du 23 mai.

Les renseignements que contiennent les tableaux I et II ont été présentés au cours de la séance plénière du 25 mai. Le projet de constitution d'un répertoire d'experts canadiens en tant que moyen peu coûteux de transfert de connaissances techniques (article 3 du tableau II) a fait l'objet d'un intérêt considérable et de fortes délibérations. Le manque total d'intérêt au sujet d'un bulletin mensuel (article 11 du tableau II) est attribuable à la reconnaissance de l'impossibilité de justifier la création d'un organisme d'accomplissement du travail nécessaire.

M. Morton a souligné l'absence d'apports scientifiques aux modèles hydrologiques. Les modèles utilisés actuellement ont pour fondement scientifique la loi de la conservation de la masse, mais le cycle hydrologique est représenté par une agglomération d'hypothèses contestables approchée de la réalité en trafiquant les coefficients pendant l'étalonnage. Les problèmes principaux résident dans la complexité des modèles, l'interdépendance des hypothèses et l'acceptation intégrale d'un modèle pour l'organisme utilisateur. Ils rendent très difficile le dépistage des hypothèses inexactes et encore plus difficile leur rectification. Par conséquent, l'inefficacité d'un modèle est d'ordinaire attribuable à l'inadéquation des données d'entrée (voir l'article 1 du tableau I) ou de l'étalonnage. Les apports scientifiques les plus valables à la modélisation hydrologique seraient:

(1) La recherche sur le cycle hydrologique et la formulation de résultats positifs de sorte qu'ils puissent être éprouvés à l'aide de données hydrométéorologiques d'usage aux fins des modèles hydrologiques. Ce genre d'apport est implicitement visé par les articles 5, 7, 19 et 21 du tableau I. Les perspectives immédiates ne sont pas encourageantes parce que les poussées de notre connaissance des interactions complexes du cycle hydrologique sont rares.

(2) La révision des hypothèses sur lesquelles se fondent les différents modèles hydrologiques. Cela est relativement facile parce que la documentation est pleine de données et de théories illustrant l'insuffisance de nombreuses hypothèses dans des circonstances particulières. Cette révision aiderait les organismes utilisateurs à choisir le modèle le plus facile à rectifier (par ex., celui qui a le moins d'hypothèses interdépendantes) ou à juger si le faible rendement d'un modèle est attribuable à l'inexactitude des hypothèses ou des données d'entrée ou à des problèmes d'étalonnage.

(3) L'établissement de méthodes d'évaluation objective du rendement des modèles. Ces méthodes seraient plus poussées que celles proposées à l'article 15 du tableau I, et aideraient les organismes utilisateurs à comparer les avantages de différents modèles et à déterminer le besoin d'améliorer le contenu hydrologique ou les données d'entrée, ou d'intensifier l'étalonnage. Les délibérations qui ont eu

lieu le 24 et le 25 mai ont inclus des reports à un document intitulé A Criterion of Efficiency for Rainfall-Runoff Models, de M. Garrick, C. Cunnane et J.E. Nash, qui a paru dans le Journal of Hydrology, n° 36 (1978), pages 375 à 381.

On a obtenu de nouveaux renseignements ayant trait aux considérations du groupe de travail depuis la fin de l'atelier. D'abord, la Commission des eaux des provinces des Prairies a réalisé une analyse de sensibilité, du type recommandé à l'article 6 du tableau I, sur le modèle de bassin versant Stanford appliqué au bassin de recherche Davin en Saskatchewan. Cette analyse est présentée aux annexes A et B au rapport de la CEPP sur la prévision des débits de la rivière Qu'Appelle jusqu'à la limite entre la Saskatchewan et le Manitoba, rapport publié en octobre 1975. On peut obtenir des copies de ce rapport et d'autres renseignements de M. Nemanishen. Ensuite, l'Institut national de recherche hydrologique réalise un projet d'amélioration des assises physiques et de l'étalonnage du système Sacramento de modélisation de bassin hydrographique, en y ajoutant des évaluations indépendantes de l'évapotranspiration effectuées selon une technique qui a été mise à l'essai dans tout un éventail de milieux. On peut obtenir des renseignements de M. Morton.

TABLEAU I

LACUNES DES CONNAISSANCES ET DOMAINES DE RECHERCHE EN MODÉLISATION DE RUISSELLEMENT

Article		Degré de priorité*
1.	Amélioration des prévisions de 24 heures et de 48 heures des précipitations.	41
2.	État exhaustif de la DGEI sur les données hydrométéorologiques nécessaires à la prévision en temps réel.	41
3.	Publication de cartes des précipitations et des couvertures nivales établies par télédétection.	40
4.	Obtention et publication par les organismes fédéraux appropriés des résultats d'analyse hydrologique nécessaires à la mise en valeur des ressources en eau du Canada.	40
5.	Recherche sur les rapports entre l'humidité des sols et le ruissellement.	39
6.	Analyse de sensibilité des modèles, par les modélisateurs des ressources en eau, sous les rapports de différents paramètres hydrométéorologiques d'entrée.	38
7.	Elaboration de méthodes améliorées de détermination des indices antécédents d'humidité du sol.	38
8.	Télédétection de l'enneigement.	38
9.	Télédétection de l'humidité du sol.	37
10.	Établissement de méthodes de détermination des précipitations surfaciques intégrées.	36
11.	Détermination de modèles pouvant être appliqués à des bassins hydrographiques particuliers.	34

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*Le degré de priorité le plus élevé possible est 42.

Article		Degré de priorité
12.	Mise sur pied de systèmes d'entretien, de mise à jour et de distribution de modèles choisis, grâce au parrainage gouvernemental.	34
13.	Mise au point de techniques et de stratégies améliorées d'acquisition de données sur l'humidité du sol et la fonte des neiges.	33
14.	Collaboration de la Direction des ressources en eau et du SEA à la planification des réseaux et à la réalisation de tous les travaux hydrométéorologiques.	33
15.	Normalisation des méthodes de comparaison objective des modèles.	31
16.	Fourniture, par le SEA, de prévisions des températures et des précipitations saisonnières.	30
17.	Amélioration ou élaboration par le SEA de prévisions météorologiques de six à dix jours.	29
18.	Poursuite de la mise au point de modèles hydrologiques probabilistes.	29
19.	Études concernant tous les effets de l'urbanisation sur les régimes hydrologiques.	27
20.	Délimitation précise des données hydrométéorologiques nécessaires à la prévision opérationnelle.	26
21.	Relevé des effets des changements climatiques prévus (pour les années 1980 et 1990) sur les régimes de sécheresse et d'inondation au Canada.	23

TABLEAU II

MÉTHODES AMÉLIORÉES DE TRANSFERT DES CONNAISSANCES TECHNIQUES

Article		En faveur*
1.	Visites consultatives.	14
2.	Ateliers.	11
3.	Répertoire d'experts canadiens (donnant les numéros de téléphone, etc.).	11
4.	Groupes de modélisation se réunissant régulièrement.	11
5.	Études mixtes Direction générale des eaux intérieures - Service de l'environnement atmosphérique - provinces (par ex., rivière Saint-Jean).	11
6.	Séance de formation sur des modèles particuliers.	10
7.	Mise en circulation par la bibliothèque d'Ottawa de résumés pertinents.	9
8.	Amélioration des voies de communication pour la consultation ascendante sur les problèmes.	9
9.	Création de groupes de coordination de la modélisation.	8
10.	Amélioration de l'emploi du WATDOC.	7
11.	Bulletin mensuel.	0

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*La cote la plus élevée possible est 14.

RAPPORT DU GROUPE DE TRAVAIL 3

Président: R.L. Pentland

Analyse stochastique et recherche opérationnelle

Le groupe de travail 3, en résumant les résultats de ses délibérations, a reconnu n'avoir traité que d'une faible proportion des travaux en cours au Canada dans le domaine de l'analyse de la fréquence des inondations. Bien qu'il se soit servi des documents présentés en tant que points de discussion initiaux, le groupe s'est efforcé de se mettre d'accord sur plusieurs questions beaucoup plus vastes que celles visées par ces documents. De plus, le groupe s'est efforcé de relier ses conclusions sommaires le plus possible aux éléments du Programme national de réduction des dommages causés par les inondations.

Les sujets traités par le groupe de travail se répartissent entre quatre catégories principales: l'analyse de fréquence à des endroits contrôlés, les endroits non contrôlés et les analyses régionales, les situations dans les grands lacs et les zones côtières ainsi que l'exploitation de réservoirs, et les interventions physiques. Le groupe a relevé les résultats nécessaires, en général, au personnel d'exécution du Programme de réduction des dommages causés par les inondations, et les besoins en recherche, soit des liens qui doivent être établis entre ce programme et les programmes de recherche. Les conclusions du groupe sont résumées au tableau 1. Le tableau 2 constitue un sommaire des conclusions les plus générales tirées par le groupe qui sont jugées importantes pour le Programme de réduction des dommages causés par les inondations mais qui ne pouvaient pas être classées avec les conclusions spécifiques. Vous trouverez ci-joint, à titre d'information, un relevé des nombres des organismes utilisant les différentes techniques d'analyse de fréquence (tableau 3).

TABLEAU 1

Conclusions sommaires

Catégorie	Résultats	Besoins en recherche
Analyse de fréquence à des endroits contrôlés	L'institutionnalisation de procédures est à éviter (selon l'expérience des États-Unis, de la Grande-Bretagne, etc.).	Il est possible d'utiliser davantage les données existantes (par ex., plus d'une crête par année).
	Il y a trop d'utilisations aveugles des mathématiques (et quelquefois d'hypothèses incorrectes sur la distribution simple, la stationarité, l'indépendance et la précision des données); les paramètres sont trop nombreux et les données, trop rares.	Il est nécessaire d'examiner les situations à probabilité combinée telles que celles qu'on trouve à des confluents, dans des estuaires, etc.
	Il y a abondance de méthodes et de compétences, mais absence de critères de sélection. Plusieurs méthodes doivent être essayées; il ne faut pas sous-estimer (ni surestimer) la valeur du jugement subjectif.	Il faut étudier les phénomènes naturels qui provoquent l'instabilité hydrologique tels que l'envasement et les embâcles de glace.
	Les données sur les écoulements à des valeurs extrêmes sont en elles-mêmes des extrapolations; il faut utiliser le plus possible les données sur les niveaux d'eau.	Il faut connaître la présence "d'échos" dans certaines circonstances et adopter des techniques pour les supprimer.
Endroits non contrôlés et analyse régionale	Le MDE devrait assumer la responsabilité du contrôle de la qualité des analyses réalisées par d'autres organismes fédéraux, et trouver les moyens d'arriver à cette fin.	La fréquence des précipitations et de la fonte des neiges peut être prévue par des modèles plus concrets applicables aux endroits non contrôlés.

TABLEAU 1 (suite)

Catégorie	Résultats	Besoins en recherche
	On a souvent tendance à perfectionner à outrance les modèles applicables aux endroits non contrôlés, c'est-à-dire qu'il y a trop de "facteurs de distorsion".	Il est nécessaire d'examiner des régions définies selon des critères physiographiques et climatiques plutôt que des régions purement géographiques.
	Bien que la méthode indiciaire soit celle qui est utilisée le plus souvent pour les analyses régionales, la régression directe gagne du terrain auprès de nombreux analystes.	Il faudrait faire intervenir des techniques de définition de fréquence des durées et des volumes.
Situations des grands lacs et des zones côtières, et exploitation de réservoirs	On reconnaît de plus en plus l'importance de la gestion des utilisations des terres par opposition à la manipulation des niveaux des eaux.	Il faut mettre au point et vérifier des modèles tenant mieux compte de l'action des vents et des vagues.
	Nous avons mis beaucoup de temps à reconnaître la complexité et l'importance des analyses sur les lacs de taille et les zones côtières.	Il faut introduire des éléments déterministes, par ex., dans les zones de marée, et le facteur de dépendance temporelle, par ex., dans le cas des grands lacs.
	Les aspects qui sont probablement les plus importants des modèles de gestion des réservoirs, sont le besoin de réalisme, la simplicité, et l'acceptation par les organismes d'exploitation.	Il faut améliorer les techniques pour comparer des facteurs (évaluation économique, nombreux intérêts ayant des importances très divergentes, aspects environnementaux, etc.) qui ne sont pas normalement comparés au cours des études de réservoir.

TABLEAU 1 (suite)

Catégorie	Résultats	Besoins en recherche
Incidences des interventions physiques sur l'analyse de fréquence	On reconnaît peu les changements physiques de plus en plus importants et leur incidence sur la fréquence des inondations (et surtout sur le drainage).	Il faut beaucoup intensifier les efforts d'élaboration de modèles fondés sur des données physiques afin d'examiner les effets du drainage, des routes, des herbes aquatiques, des pratiques d'exploitation forestière.
	La recherche en matière d'analyse statistique des interventions a progressé, mais n'a que très peu été appliquée aux activités de gestion des eaux du Canada.	Il faut transposer les résultats obtenus sur de petits bassins de recherche à des cours d'eau plus importants et à des régions plus peuplées.

TABLEAU 2

Observations générales

1. Bien que l'analyse de fréquence soit la source d'erreurs la plus importante en cartographie des plaines inondables, elle est probablement la moins sujette à des problèmes juridiques. Cependant, il ne faudrait pas pour autant imputer les négligences de cartographie et d'hydraulique aux inexactitudes de cette analyse.

2. Il est probable que 80% des problèmes relatifs aux situations des grands lacs et des zones côtières soient attribuables à l'érosion. Il importe de reconnaître les différences des politiques applicables aux inondations et à l'affouillement aux chapitres notamment de l'aide aux sinistrés, de l'efficacité des ouvrages de protection, etc. Il convient d'étudier le besoin d'appliquer les principes de réduction des dommages causés par les inondations aux risques d'affouillement, mais le bien-fondé de pareilles mesures du point de vue fédéral n'est pas encore clair.

3. Il est nécessaire de mieux définir la protection contre les inondations.

4. Le moment est venu d'organiser une réunion de tous les membres, fédéraux et provinciaux, du comité technique pour qu'ils échangent leurs expériences, et d'assurer toute la cohérence souhaitable du programme national.

TABLEAU 3

Techniques d'analyse de fréquence

Distribution préférée	Nombre d'organismes utilisateurs
Normale	2
Gumbel 1	15
Pearson 3	5
Log Pearson	7
Log Normal	9
3-P Log Normal	7
Kritskii-Menkel	1
Nombre d'organismes	23

OBSERVATIONS DU PRÉSIDENT DE L'ATELIER

E. Watt

1. ORGANISATION ET PARTICIPATION

1.1 Arrangements et services de soutien

MM. H.R. Eisenhauer et D.W. Farley se sont admirablement bien acquittés de tous les détails pratiques de l'atelier, notamment du choix de l'endroit, de la date, du calendrier des activités, de son bon déroulement, du tirage préliminaire des communications, etc. Ces dernières ont été distribuées suffisamment longtemps à l'avance, les salles de réunions convenaient très bien au besoin, et le calendrier s'est révélé des plus appropriés.

1.2 Présidents de groupe et auteurs

Les présidents des trois groupes de travail ont assumé leurs fonctions avec compétence. D'autre part, chaque auteur a fourni un effort notable pour ne pas dépasser les cinq minutes qui lui étaient allouées pour résumer sa conférence, à la séance d'ouverture. Cela a assuré un bon départ aux travaux.

1.3 Participants

Toutes les régions, les deux instituts de recherche et l'administration centrale étaient représentés. On comptait de plus des personnes venant d'organismes provinciaux, d'universités et de firmes-conseils. Le "mélange" était bon. On aurait toutefois pu s'attendre à une participation plus grande de l'INRH.

1.4 Connaissance du Programme de réduction des dommages causés par les inondations

Il a été assez surprenant de constater que certains participants semblaient ignorer le détail de ce programme, surtout à un atelier consacré à la modélisation à des fins de réduction des dommages dus aux inondations.

2. LA MODÉLISATION

2.1 Conclusion

Il semble que les communications présentées par le personnel de la DGEI reflétaient assez fidèlement les travaux de cette dernière sur la modélisation relative à la réduction des dommages causés par les inondations.

2.2 Pertinence

La majorité des travaux de modélisation exposés dans les communications étaient pertinents à la réduction des dommages dus aux inondations. Toutefois, dans certains cas, ils étaient inopportuns.

2.3 Efficacité et coordination

Bien qu'on ait noté certains exemples exceptionnels de coordination et de coopération entre les régions et l'administration centrale, il semble que les travaux de modélisation se soient déroulés en général sans coordination. Une telle attitude semble, au premier abord, inefficace et susceptible de donner lieu à un chevauchement des efforts. Ce point a été soulevé par les gens du groupe 1, lors d'une discussion sur les modèles dynamiques des canaux ouverts. L'observation peut s'appliquer aux travaux d'analyse de la fréquence des crues.

2.4 Recherche et développement vs opérations

On s'attendait à ce que le personnel des régions s'intéresse davantage aux problèmes opérationnels et celui de la DGEI (administration centrale), de l'INRF et de l'INRH, davantage à la recherche et au développement tout en gardant à l'esprit les problèmes opérationnels liés à la pratique.

Mais seul le personnel de l'INRF et de l'administration centrale de la DGEI a répondu à cette attente (celui de l'INRH n'a présenté aucune communication). Bien que la plupart des communications présentées par le personnel des régions ait traité de problèmes opérationnels d'ordre pratique, un nombre important d'entre elles a porté sur des activités de recherche et de développement qui, normalement, sont effectuées par un institut de recherche. Même si cette approche repose probablement sur des raisons fort valables, il reste qu'elle est inappropriée et ne donne pas lieu, en général, à une très grande qualité de recherche et de développement.

3. LES BESOINS

3.1 Besoins techniques

Les participants ont dressé les listes des lacunes de connaissances, des besoins en recherche, etc., spécifiques à chacun des trois domaines traités: (1) modélisation hydraulique des canaux ouverts; (2) modélisation hydrologique; (3) analyse stochastique et recherche opérationnelle. Ces listes figurent dans les rapports des présidents de groupes. Il y aurait lieu de les revoir et d'en classer les éléments par ordre de priorité, d'effort de recherche requis, et de planifier la recherche à effectuer. Il est important de souligner le besoin de modèles adéquats pour l'analyse des inondations dues aux embâcles. L'acquisition de cette information est urgente en ce qui concerne les programmes de cartographie en cours. On ne peut espérer qu'une aide très faible de l'étranger et, par ailleurs, les fonds et le personnel dont nous disposons pour étudier ce problème sont fort restreints.

3.2 Communication et transfert des connaissances technologiques

Les groupes (1) et (2) ont discuté des problèmes de communication et ont proposé de meilleures méthodes de transfert des connaissances technologiques. Il est clair que ces problèmes sont d'ordre majeur et doivent être résolus sur-le-champ. Un excellent exemple de cette nécessité s'est manifesté pendant l'atelier même. Un auteur a déclaré qu'il lui était impossible d'appliquer à ses données un modèle mis au point par le personnel de l'administration centrale. En fait, ses données n'avaient pas été classées selon les exigences du modèle. Il est ressorti que ni l'auteur (un "usager") ni le modèle n'était fautif. Les auteurs du modèle avaient préparé et distribué un bon guide destiné à l'usager, mais comme ils avaient depuis été affectés ailleurs, plus personne, à l'administration centrale, n'était en mesure de répondre rapidement aux demandes de renseignements des usagers.

Si les méthodes de transfert technologique ne sont pas améliorées, les résultats des programmes de recherche des instituts et de l'administration centrale ne seront que faiblement utilisés par les régions. De plus, il est important que les groupes des régions profitent d'un lien efficace avec les groupes de recherche afin de renseigner ces derniers sur les problèmes de la "vraie vie".

3.3 Besoins organisationnels

Il est évident que dans un organisme où les travaux de recherche et de développement, limités à un aspect particulier des ressources en eau, sont effectués par au moins neuf unités (5 régions, l'INRF, l'INRH, la DRE (Hull), la DPGE (Hull)), l'effort fournit pour assurer l'opportunité, l'efficience et l'efficacité des travaux doit être considérable. Les problèmes suivants ont été identifiés:

(i) absence de coordination, chevauchement des efforts (2.3);
(ii) contrôle irrégulier de la qualité (2.4);
(iii) manque d'identification des autorités compétentes;

(iv) communication et méthodes de transfert technologiques imparfaites (3.2); et

(v) absence de stratégie d'ensemble évidente qui fixe les rôles de chacun et les liens entre les instituts de recherche, les universités, les groupes de l'administration centrale, le secteur privé et les régions, en matière de recherche et de développement.

4. RÉSUMÉ

En résumé, l'atelier a été bien planifié et bien organisé. La participation s'est révélée excellente à tout moment et les auteurs des documents méritent tous nos éloges quant à la rapidité et à l'efficacité dont ils ont fait preuve dans la préparation et la présentation de leurs exposés.

Il est clair que tous les objectifs de l'atelier, déjà énumérés, ont été atteints. Toutefois, bien que cette étape puisse être considérée comme essentielle, elle ne suffit pas à assurer la pleine réalisation des responsabilités de la DGEI, en ce qui a trait à la modélisation à des fins de réduction des dommages causés par les inondations. Il faut continuer les travaux de recherche et de développement afin de combler les lacunes de connaissances, en particulier dans le cas des inondations dues aux embâcles, et trouver de nouvelles méthodes qui assurent le transfert des résultats de recherche aux services opérationnels.

En plus de produire les effets voulus, l'atelier a permis de déceler des problèmes organisationnels néfastes à l'efficacité des travaux de recherche et de développement, en matière de modélisation. Par conséquent, même si l'atelier a porté sur un aspect particulier des ressources en eau, il est permis de croire que des problèmes organisationnels similaires existent peut-être dans d'autres domaines de recherche relatifs à ces ressources.

ÉVALUATION DE L'ATELIER

H.R. Eisenhauer

Tous les participants ont reçu un formulaire leur permettant de donner leur avis sur l'atelier. Des 46 participants, 25 ont répondu aux questions des organisateurs. Voici une analyse de leurs observations.

1. Installations

L'atelier a eu lieu au Centre de conférence de Place Vincent Massey. Le premier jour, tous les participants étaient rassemblés dans la salle Churchill, qui peut contenir jusqu'à 80 personnes. Les tables étaient éparpillées plus ou moins au hasard dans toute la salle, avec trois ou quatre chaises placées à chacune d'elles de façon que les participants puissent faire face à l'avant de la salle.

Le deuxième jour, on s'est divisé en trois groupes de travail, lesquels ont été installés dans les salles Churchill, Saguenay et Mackenzie. Dans ces deux dernières, le plus grand nombre possible de participants étaient assis autour d'une même table, alors que les autres étaient assis tout près. Seize personnes pouvaient s'asseoir autour de la table de la salle Saguenay et douze, dans la salle Mackenzie. Les groupes de travail logés dans ces salles comptaient respectivement 18 et 14 participants. Le troisième jour, à la séance plénière, tous les participants se sont réunis de nouveau dans la salle Churchill.

Toutes les salles étaient équipées d'un projecteur pour diapositives de 35 mm, d'un rétroprojecteur, d'un tableau noir et d'un tableau de conférence.

Observations

Presque tous les répondants se sont dits satisfaits de la Salle Churchill pour les séances plénières, mais plusieurs ont observé que la salle Mackenzie était trop petite, pour le groupe de travail. La plupart n'ont rien à redire à propos des sièges, mais un participant a trouvé qu'ils étaient trop tassés, dans la salle Churchill; un autre a dit avoir eu du mal à voir les diapositives. L'équipement a paru satisfaisant à tout le monde.

2. Présentation

Le premier jour de l'atelier (plénière), trois domaines ont été abordés, qui regroupaient de trois à cinq sujets chacun. Chaque sujet a fait l'objet d'un à quatre documents de travail, qui avaient été distribués bien avant l'atelier. Les auteurs des documents en ont présenté un bref résumé (n'excédant pas cinq minutes). Après la présentation d'un groupe de résumés, les auteurs ont pu répondre aux questions ayant trait essentiellement au sujet en question. La discussion ne devait pas non plus durer plus de cinq minutes par document.

Le deuxième jour, trois groupes de travail ont été formés, pour étudier chacun un domaine. Les participants, qui devaient avoir lu les documents au complet, ont été appelés à contribuer aux travaux du groupe auquel ils avaient été assignés. Chaque auteur a, pendant 10 à 15 minutes environ, apporté des précisions sur son document, ajoutant des détails et faisant ressortir les problèmes et difficultés. Ensuite, sous la gouverne du président du groupe de travail, on a échangé sur les problèmes et les lacunes de connaissances, puis on a formulé des recommandations axées sur la résolution, la recherche ou la gestion.

À la plénière de fermeture, le troisième jour, chacun des présidents de groupe a fait le bilan des discussions et recommandations de son groupe, à la suite de quoi a eu lieu une discussion générale. Le bilan et la discussion pour chacun des groupes devaient se faire dans environ 45 minutes.

Il y a eu deux pause-café de 20 minutes par jour, dans la matinée et dans l'après-midi. De plus, une réception non officielle (chacun payait ses consommations) a été organisée à la fin de la première journée pour "briser la glace" et favoriser les contacts.

Observations

Quatre-vingt-dix pourcent des répondants ont trouvé que le nombre de sujets touchés au cours de l'atelier était approprié; deux personnes ont dit qu'il y en avait trop. Selon un participant, les présentations étaient trop familières; toutefois, tous les autres se sont dits satisfaits du degré de formalité. Près de 20% de ceux qui ont répondu auraient aimé que le temps accordé à la présentation des documents soit plus long et environ 10% ont fait la même remarque en ce qui a trait aux périodes de discussion. Quatre personnes ont trouvé que les discussions ne devraient se faire qu'à la fin de chaque demi-journée et environ autant ont eu l'impression que les discussions n'étaient pas spontanées et ne permettraient pas à chacun de s'exprimer. Quelqu'un a avancé que cela était dû au fait que le groupe était trop important et diversifié alors que, selon un autre, c'est parce que trop d'auteurs ont "vanté leur travail" plutôt que d'exposer les difficultés et problèmes rencontrés. La plupart des participants s'accordent à dire que les présidents ont fait du bon travail; certains ont affirmé que les présidents pourraient faire mieux encore s'ils rencontraient les auteurs avant les séances de travail afin d'identifier les questions sujettes à controverse, en faisant auparavant étudier les documents par des experts, pour que ceux-ci établissent autour de quelles questions pourraient se faire les discussions, et en faisant en sorte que tous les documents soient touchés de façon équitable pendant la période accordée pour la présentation et la discussion. Un répondant s'est plaint que le président de son groupe de travail a trop souvent fait valoir son point de vue au cours de la discussion. Même si la réception aurait pu être plus longue, on semble, de façon générale, satisfait des possibilités qu'elle offrait en matière d'échanges individuels.

3. Documentation

Tous les participants ayant un rôle particulier à jouer (auteurs et présidents) ont reçu, environ 10 semaines à l'avance, une description détaillée de l'atelier comprenant l'objectif, la formule et le programme de l'atelier ainsi que les responsabilités des participants. Tous les auteurs ont été priés de préparer un document de travail (d'un maximum de 10 pages) ainsi qu'un bref résumé. La plupart des documents ont été reçus tôt et ont pu être envoyés à tous les participants à peu près un mois avant la tenue de l'atelier. Chaque participant devait plus particulièrement lire les documents que le groupe de travail auquel il était assigné étudierait en détail.

Observations

Seulement deux des répondants ont trouvé que le but de l'atelier n'était pas clair et n'a pas été atteint. Près de 90% des répondants ont dit que l'atelier avait touché l'ensemble de la question et tous, sauf un, ont trouvé que les sujets étaient, de façon générale, pertinents par rapport au thème de l'atelier. D'aucuns se sont dits d'avis que les documents auraient pu être davantage axés sur la réduction des dommages causés par les inondations et que l'existence d'un groupe d'étude des documents aurait pu donner lieu à une meilleure sélection. Plus de 90% des répondants ont trouvé que la documentation préliminaire (documents de base) était appropriée, traitait de façon satisfaisante des sujets et présentait une certaine utilité. Deux personnes ont noté qu'un grand nombre d'auteurs n'avaient pas suivi les directives en ce qui a trait à la longueur des documents et à l'identification particulière, entre autres, de problèmes, de questions, de lacunes de connaissances, qui auraient pu encourager une discussion utile. Près de 90% des personnes qui nous ont répondu étaient satisfaites de la qualité des présentations. Un participant a remarqué que la plupart des auteurs avaient tendance à "resservir" leur document, lors de la présentation de la deuxième journée, alors qu'un autre n'a pas aimé que l'on donne aux présentations un caractère général, la première journée, pour les traiter de façon plus particulière, la deuxième journée. Une personne sur sept a trouvé qu'on faisait trop peu appel à du matériel visuel, lors des présentations, et quelqu'un a mentionné qu'il aurait été bon de mettre à la disposition des participants des transparents et des plumes afin de présenter des résumés à l'aide de rétroprojecteurs. Tous les répondants sont d'accord quant à l'utilité de la publication du compte rendu de l'atelier. Trois personnes ont souligné que les recommandations formulées par les groupes de travail (notées par les présidents de groupe) devraient être présentées à la haute direction de la DGEI.

4. Participants

Le thème de l'atelier, de même que les documents particuliers et les auteurs, ont été recommandés par un comité spécial d'organisation et soumis à l'approbation du comité des directeurs de la DGEI. C'est ce qui a permis de déterminer quels seraient les "animateurs" de l'atelier. Les directeurs des bureaux régionaux et de l'administration centrale ont en outre désigné des participants additionnels, et leur choix a été soumis à l'approbation du comité des directeurs. Il y avait 46 participants, dont 30 ont joué un rôle actif à titre d'auteur ou de président (voir la liste des participants). Trente-trois personnes étaient de la DGEI (17 de bureaux régionaux, 11 de l'administration centrale, 5 des instituts de recherche), deux venaient d'autres organismes fédéraux, six, d'organismes provinciaux, quatre, de diverses universités, et une, du secteur privé.

Observations

Dix pour cent des répondants ont trouvé qu'il y avait trop de participants, et autant ont trouvé qu'il n'y en avait pas assez. Vingt pour cent des répondants se sont dits d'avis qu'il aurait moyen d'améliorer le choix des participants en augmentant la participation de spécialistes reconnus (experts-conseils et organismes provinciaux) qui travaillent véritablement dans le domaine.

5. Généralités

Tous les répondants ont trouvé que l'atelier permettait un transfert de connaissances. Une personne a trouvé que la séance de travail approfondi en groupe avait été la plus utile et que les séances plénières ne présentaient aucune importance particulière. Une autre a recommandé l'existence de brefs cours donnés par des spécialistes sur des sujets précis.

L'atelier a été enrichissant, du point de vue personnel, pour tous les répondants. Deux d'entre eux ont noté que l'atelier avait eu une importance particulière en raison des contacts qu'ils avaient pu y établir avec d'autres personnes travaillant dans le domaine.

Le tiers des répondants a fait des commentaires très favorables en ce qui a trait à la qualité de l'organisation, de la planification et du fonctionnement de l'atelier.

6. Recommandations

Quatre-vingt-cinq pour cent des répondants ont fait des recommandations visant l'amélioration des ateliers à venir. Ces recommandations sont résumées ci-dessous. Elles proviennent, la plupart du temps, d'un seul individu. Lorsque la recommandation a été faite par plus d'un participant, le nombre de personnes est indiqué entre parenthèses.

6.1 Installations

- L'atelier devrait avoir lieu dans une salle qui permette à tous les participants de voir sans difficulté les diapositives et les autres documents visuels.
- Quelqu'un devrait être là pour faire fonctionner les projecteurs.
- Il devrait y avoir des transparents et des crayons, pour que l'on puisse se servir des rétroprojecteurs.
- Il faudrait favoriser les réunions autour de petites tables.

6.2 Formule

- Il serait bon que l'on alloue plus de temps à la présentation des documents et à la période de discussion qui suit (5). Cela pourrait se faire si l'on restreignait le domaine et si l'on choisissait moins de sujets (4).
- Une période de discussion devrait suivre la présentation de chaque document (2) et le président devrait donner une vue d'ensemble, à la fin de la journée.
- Les présidents devraient voir à ce que le même temps soit accordé à toutes les présentations et discussions.
- Les présidents devraient rencontrer les auteurs avant l'atelier, afin d'identifier les questions qui risquent de faire l'objet de controverses.
- Il serait bon que les présidents de groupe se rencontrent afin d'établir une approche commune en ce qui a trait aux séances et à leurs responsabilités.
- Une note biographique des auteurs devrait accompagner les documents de travail ou leur servir d'introduction.
- Il serait bon que les présidents voient à ce que les questions et les discussions entrent dans le cadre de l'objectif de l'atelier.

- Une importance particulière devrait être accordée aux présentations détaillées et aux périodes de discussion (4); en outre, les présidents de groupe devraient donner un résumé de tous les documents la première journée (une ou deux heures). (Note: Par contre, un participant a remarqué que la première journée était des plus instructives, alors qu'un autre a affirmé que la troisième journée était très réussie.)

- La rencontre sociale devrait avoir lieu le dernier soir de l'atelier, dans un hôtel - la période d'une heure de bar ouvert était trop courte.

6.3 Documentation

- Il faudrait que tous les auteurs suivent les directives concernant la longueur des documents de travail (10 pages) et le contenu (identification de problèmes et de lacunes de connaissances, les améliorations qu'il aurait été possible d'apporter, etc.) (6).

- On devrait inviter les auteurs à présenter des documents, lesquels seraient étudiés de façon critique par des experts, puis revus conformément aux lignes directrices, pour finalement faire l'objet d'une sélection, en fonction de leur pertinence, par rapport au sujet de l'atelier (5).

- Des documents devraient être produits pour l'atelier en réponse aux documents de travail, afin de faire état de toutes les opinions et de promouvoir une discussion animée et enrichissante (2).

- On devrait encourager tous les conférenciers à se servir de matériel visuel.

- Les recommandations préparées par les présidents de groupe devraient être mises en relief dans le compte rendu, et non simplement résumées, car elles risqueraient alors de passer inaperçues.

- Les recommandations issues de l'atelier devraient être transmises directement à la haute direction pour qu'elle les étudie, prenne des mesures à leur sujet et y donne suite (3).

- Dans les présentations orales, les auteurs devraient s'efforcer de présenter de façon concise les éléments techniques.

6.4 Participants

- Il faudrait voir à ce que tous les participants aient la compétence technique nécessaire et apportent leur collaboration, particulièrement en ce qui concerne ceux qui travaillent véritablement dans le domaine (4).

- Il faudrait s'efforcer d'obtenir la participation de spécialistes des gouvernements provinciaux et du secteur privé (4).

- Chaque groupe de travail devrait comprendre moins de participants.

- Chaque groupe de travail devrait comprendre plus de participants.

6.5 Généralités

- Il devrait y avoir plus d'ateliers de ce genre.

- Il serait bon que l'on envisage la possibilité que des spécialistes de domaines précis donnent de brefs cours.

- Il faudrait envisager la possibilité de tenir des réunions pour des "groupes d'usagers".

- Des courts ateliers devraient être organisés comme mesure de relance, afin de traiter de sujets particuliers comme la modélisation hydrodynamique unidimensionnelle, l'analyse des interventions, l'étude prospective des bassins versants.

- Il serait bon que l'on recueille, à la fin de l'atelier, les observations relatives à l'atelier.

C. RÉSUMÉS DES DOCUMENTS DE BASE

SIMPAK for River Basin Studies

J.C.Y. Lee et R.O. Lyons

Dans plusieurs études faites dans la Région du Pacifique et du Yukon, des modèles du cheminement de l'écoulement ont servi à déterminer les effets, en aval, de divers projets de réservoir ou de dérivation. Dans les études antérieures menées dans la région, le modèle SSARR a été utilisé pour mener ce genre de recherche. Seule une partie du modèle était cependant nécessaire. Pour arriver aux mêmes résultats, il aurait suffi d'employer les méthodes du calcul du cheminement du SSARR et d'établir un plan de programme distinct pour le traitement des données. C'est ainsi qu'on a développé le programme de simulation SIMPAK pour la modélisation des cours d'eau, dont le bureau de Vancouver, de la DGEI, s'est servi pour étudier certains bassins. On l'a utilisé pour calculer la propagation des crues, pour régulariser le débit de réservoirs, pour prévoir l'écoulement critique des crues et pour effectuer d'autres travaux nécessitant le traitement de données sur l'écoulement. Les prévisions de l'écoulement critique des crues ont, en particulier, été établies afin de moduler quotidiennement les réservoirs pour la défense contre les crues. Bien que simple, la technique mise au point pour faire ces prévisions constitue un instrument utile et maniable. Dans le présent document, ont décrit les composantes du programme, confronté au modèle SSARR, et l'on en donne certaines applications.

Development of a Real-Time Operational Model for the Ottawa River - a Progress Report

G.A. Nix

En mai 1974, le Canada et le Québec ont signé un accord en vue d'entreprendre des recherches sur les problèmes d'inondation et d'étiage dans la région de Montréal. Dans le rapport, il a été établi que la seule façon de réduire les dangers d'inondation dans cette région consiste à agir le plus efficacement possible sur les nombreux réservoirs du bassin des Outaouais. À la suite des recommandations énoncées dans le rapport, le Comité de planification de régularisation du débit de la rivière des Outaouais a été créé en février 1977. Le Comité a mis sur pied un sous-comité chargé d'établir des critères en vue de procéder à cette régulation, en tenant compte de la production hydroélectrique, de la défense contre les crues, de la navigation, des problèmes d'étiage, des normes de qualité de l'eau et des loisirs. Un groupe de travail a ensuite été formé pour établir un modèle mathématique visant à régulariser ce débit. Le présent rapport fait le point sur les travaux de modélisation.

HEC-2 Application at Moose Jaw - Input Requirements and Model Performance

R.D. Hofer

Le 8 mars 1978, la DGEI et le ministère de l'Environnement de la Saskatchewan convenaient d'entreprendre des recherches en hydraulique en vertu de l'accord Canada-Saskatchewan visant à réduire les dommages causés par les inondations. Le marché initial prévoyait l'établissement de laisses de crues au 1/100 et au 1/500 le long des ruisseaux Thunder et Spring, de la rivière Moose Jaw, ainsi qu'un canal d'évacuation des crues le long de cette dernière. Le modèle HEC-2 a été utilisé pour modéliser l'écoulement. Le présent document décrit la marche suivie, définit les problèmes de modélisation et donne les résultats obtenus.

Mathematical Modelling of the Fraser River System

M. Sydor et P. Zrymiak

Le système complexe de canaux du bas Fraser a été étudié à l'aide d'un modèle hydrodynamique unidimensionnel aux différences finies. Les données d'entrée, recueillies en 1962 et en 1976 par le ministère des Travaux publics, concernaient environ 700 sections transversales. Les premiers succès ont été enregistrés à l'aide du modèle en fixant le débit à 350 900 pi^3/s le 12 juillet 1976 et en simulant le 4 juin 1976 le comportement du fleuve dont le débit était alors de 277 500 pi^3/s. Avec des résultats aussi encourageants, les recherches sont poursuivies. L'étude portera sur diverses périodes de l'année afin de déterminer le degré de justesse possible pouvant être atteint avec ce modèle.

Application of the DWOPER Model to the Lower Saint John River

H.S. Belore, A.K. Bhattacharyya et C.T. Bishop

Pour étudier le cours inférieur de la Saint-Jean, on s'est servi d'un modèle de la propagation des crues, appelé DWOPER (acronyme signifiant modèle opérationnel d'ondes dynamiques). Le modèle, fruit des derniers travaux de modélisation de l'écoulement non permanent, a été choisi après que l'on eût passé en revue plusieurs des techniques pouvant être appliquées. Le modèle tient compte en particulier des effets, en aval, des chutes Reversing et du ruissellement sur la transcanadienne vers de grandes zones d'emmagasinement dans la région des Grands lacs. Une fois le modèle étalonné, on a procédé à des simulations qui ont bien reproduit le niveau des crues antérieures. Par la suite, on a déterminé le profil du niveau des crues se répétant à des intervalles de 20 et de 100 ans le long du cours inférieur de la rivière Saint-Jean, entre McKinley Ferry et Lower Jemseg.

Application of Finite Element Method in River Network Hydraulics

S.M.A. Moin

Dans le présent document, on décrit un modèle aux différences finies du régime d'écoulement non permanent dans les réseaux de canaux ouverts. Du point de vue mathématique, l'hydraulique de l'écoulement non permanent et unidimensionnel des canaux se définit par deux équations différentielles partielles, non linéaires, de type hyperbolique, et se résout par l'adoption d'éléments linéaires.

Une méthode de prédiction et de correction a servi à résoudre les équations non linéaires. Pour évaluer les capacités de simulation du modèle informatisé, on a déterminé le profil permanent du Saint-Laurent. On a procédé à l'analyse de sensibilité à l'échelle réduite pour étudier l'incidence de la variation du paramètre de pondération.

En comparant les réponses du modèle aux événements observés ou aux valeurs calculées, ou aux deux, on est à même de mesurer l'efficacité du modèle du point de vue de ses capacités de simulation, de l'adoption de paliers plus élevés dans le temps, de la conservation de la masse, de la flexibilité de l'opération et du traitement de configurations complexes du réseau.

Mathematical Modelling of Unsteady Flows in Mobile Boundary Channels

B.G. Krishnappan

L'auteur décrit un modèle numérique résolvant les équations de l'écoulement non permanent d'un canal aux limites mobiles. Le modèle peut prédire les caractéristiques de l'onde de crue ainsi que les répercussions que l'activité anthropique dans les bassins auront sur l'environnement. Le modèle a été mis à l'essai dans des conditions de laboratoire. On a observé une concordance relativement bonne entre ses prévisions et les mesures prises en laboratoire.

Ice Jamming Concerns and Operational Solutions

E.R. Langley

L'examen des cas antérieurs d'embâcles et les données techniques limitées dont on dispose sur le sujet montrent qu'il est nécessaire d'établir des lignes directrices en vue d'inclure les effets des embâcles dans le travail de cartographie des risques d'inondation et qu'il est encore plus urgent de recueillir des renseignements fiables sur les embâcles. Entretemps, on propose que chaque emplacement soit considéré distinctement afin de déterminer quelle est la meilleure façon de tenir compte du phénomène des embâcles dans ce genre de cartographie.

Hydraulic Aspects of Ice Jamming

G. Tsang

Les facteurs physiques influant sur la formation des glaces et les embâcles sont passés en revue. Bien qu'il soit possible de prédire la survenue d'embâcles avec une certaine précision, il reste encore de nombreux problèmes à résoudre parmi lesquels figurent les caractéristiques des embâcles, l'instabilité de la couverture de glace, l'évolution des embâcles en mouvement et leur résistance au courant.

Meteorological Inputs for Hydrologic Models

R.G. Lawford

Les précipitations et la température sont les principaux facteurs du cycle hydrologique. Le SEA fournit des données historiques et en temps réel tandis que les centres météorologiques régionaux et canadien établissent des prévisions concernant les précipitations et la température. L'auteur rapporte les nouveaux progrès techniques et leur application aux modèles hydrologiques et donne un bref aperçu des modifications que l'on pourrait apporter à ces modèles pour représenter de façon plus réaliste les phénomènes météorologiques. Des recommandations sont faites en vue d'améliorer les modèles hydrologiques et leur application au système de prévention des dommages dus aux inondations.

The Need for a Regional Unit Hydrograph Analysis of Ontario

D. Kimmett

Une bonne part de la défiance manifestée à l'endroit des études hydrologiques antérieures vient du fait que chacun comprend différemment la théorie des hydrogrammes unitaires et préconise une application particulière. L'auteur propose que la Division des relevés hydrologiques du Canada procède aux vérifications maison jugées nécessaires et à la publication d'hydrogramme(s) unitaire(s) de façon continue et selon les besoins de certaines stations hydrométriques. Les récents progrès techniques, le besoin en général de normes en hydrologie et le fait que l'on dispose de suffisamment de données sont autant de raisons pour mettre actuellement en branle ce genre de programme. Plusieurs utilisateurs de ces données consultés récemment de façon officieuse se sont montrés favorables à l'idée. L'auteur précise ce qu'il faudrait publier.

Runoff from Developing Areas

J. Marsalek

Le ruissellement des zones en voie d'urbanisation revêt un intérêt particulier pour ce qui concerne la modélisation des crues étant donné que l'aménagement foncier modifie considérablement l'hydrologie des bassins. Au cours des quinze dernières années, plusieurs modèles simulant le ruissellement en provenance de pareilles zones sont apparus; la plupart d'entre eux ont désormais atteint le stade opérationnel. Leur développement et leur perfectionnement, actuellement en cours, portent en particulier sur l'évaluation des effets de l'urbanisation sur le régime hydrologique des bassins versants, sur la modélisation de l'infiltration des eaux et des dommages dus aux inondations.

Flood Forecasting in the Saint John River Basin - Mathematical Modelling Activities

D.A. Smith

L'auteur étudie l'établissement d'un système de prévision des crues dans la partie du bassin de la Saint-Jean du côté du Nouveau-Brunswick. Réalisée suivant un ordre chronologique, son étude débute au moment où il n'existait aucun système du genre, fait état des premières tentatives d'étalonnage du modèle SSARR pour le bassin et de prévision au cours des crues record de 1973 jusqu'à ce qui se fait aujourd'hui. Il s'attache particulièrement à présenter des travaux de modélisation mathématique et à rappeler les efforts récents en vue d'améliorer le système de prévision fonctionnant en vertu du programme que le Canada et la province ont établi en vue de réduire les dommages dus aux inondations. Il présente les principaux points de préoccupation du sous-comité technique de prévisions des inondations et traite des activités actuellement en cours ou prévues en vue de surmonter certaines de ces difficultés.

Data Network Design for Watershed Forecasting

D.C. Ambler

L'auteur décrit le réseau de collecte de données nécessaire pour prévoir les crues de la Saint-Jean notamment. Il est question des sources d'erreur de jaugeage et de leur incidence sur les réponses et les prévisions données par le modèle hydrologique.

Ottawa River - Comparison of the Hydro Québec and SSARR Models

G.E Brown

Pour confronter les modèles du débit entrant naturel d'Hydro Québec et SSARR, l'auteur a analysé les prévisions qu'ils ont établies pour le débit d'écoulement de cinq bassins de la rivière des Outaouais. Dans les cinq ans, le modèle d'Hydro Québec s'est, en général, révélé supérieur. En ce qui concerne les prévisions établies pour dix jours, le modèle SSARR a donné des résultats de plus en plus erronés. Quant à celui d'Hydro Québec, les écarts observés d'un jour à l'autre ont été minimes. L'auteur fait remarquer que le modèle d'Hydro Québec, qui s'applique à des bassins assez grands, prévoit que la moyenne du débit entrant quotidiennement atteint, au bout de sept jours, les réservoirs situés à l'extrémité de chaque bassin. Par conséquent, on ne peut se fier à ce modèle quand il s'agit de bassins jaugés hydrométriquement étant donné que les cinq dont il est question ont été étalonnés à partir de données informatisées sur le débit entrant des réservoirs.

Short-Term and Long-Term Hydrometeorological Epochs

W. Nemanishen

Du point de vue scientifique, de plus en plus de preuves tendent à démontrer que des facteurs autres que terrestres, comme l'activité solaire et les cycles lunaires, influent sur les circulations atmosphérique et océanique qui, à leur tour, concourent à provoquer, dans certaines parties de l'hémisphère nord, des sécheresses et des inondations de longue durée. L'auteur résume les recherches de plusieurs éminents scientifiques du monde pour montrer comment leurs découvertes peuvent servir à faire avancer les recherches de la DGEI en vue de lutter contre les inondations dans l'Ouest du pays. Il montre, à partir d'un cas, qu'il est difficile de déterminer avec justesse les périodes de retour des inondations se produisant rarement et qu'il est utile, dans ces cas, de recourir à la modélisation de l'écoulement.

The Three Parameter Lognormal Distribution Applied to Regional Flood Frequency Analysis by the Index Flood Method

R. Condie

L'auteur signale que l'on a recours assez souvent à la méthode d'indice des crues pour analyser leur fréquence par régions quand, aux stations de cours d'eau non jaugés, il faut évaluer l'écoulement des crues selon certaines probabilités. L'emploi de cette méthode suppose habituellement que la distribution 1 de Gumbel donne une assez bonne représentation de la fréquence des crues dans une région homogène; le test d'homogénéité le plus courant se fonde sur l'échantillonnage à partir de cette distribution. Des réserves ont été exprimées quant à la valeur de la distribution 1 de Gumbel, mais le recours à d'autres distributions a sans doute été impossible vu l'absence de tests d'homogénéité appropriés. Au Canada, la distribution log-normal à trois paramètres est généralement supérieure à la Gumbel 1. L'auteur donne les raisons de ce choix et décrit le test d'homogénéité qui s'y rattache. Il est aussi question des avantages et des désavantages de la méthode d'indice des crues par rapport à la méthode de régression de la direction sur une crue de probabilité 1/T.

Flood Frequencies by the Kinematic Wave Model

R.O. Lyons

La Région du Pacifique et du Yukon, de la DGEI, s'est servie du modèle de la cinématique des ondes comme l'une des méthodes d'estimation des crues maximales possibles dans l'étude du pipeline de la route de l'Alaska. Dans ce document, l'auteur donne brièvement les équations fondamentales sur lesquelles se fonde le modèle et décrit la façon dont il a été étalonné pour le sud du Yukon. Le modèle constitue un bon instrument d'estimation directe de la fréquence des crues dans les bassins non jaugés. Il nécessite cependant de sérieux efforts d'analyse des données météorologiques, mais une fois ce travail fait, les courbes de distribution de fréquences peuvent être tracées rapidement, à peu de frais et directement.

Flood Frequency Analysis by Power Transformation

R.S. McAully

Dans ce document, l'auteur présente une technique d'analyse de la fréquence des crues qui consiste à ajuster la distribution normale des probabilités à des séries de valeurs annuelles des crues après que l'on a procédé à ce que l'on a appelé une "transformation de puissance". L'auteur décrit le programme informatique grâce auquel on procède à cette transformation et il présente certains résultats de l'analyse de 35 cours d'eau de la Colombie-Britannique. Cette méthode de transformation, très facile d'application, est, en général, en parfait accord avec l'habituelle méthode log-Pearson III.

Flood Frequency Analysis Concerns in the Atlantic Region

J.D. Keefe

L'auteur résume certaines des difficultés techniques éprouvées dans la région de l'Atlantique au cours de la cartographie des risques d'inondation, les façons de les surmonter et précise les secteurs qui nécessitent davantage de développement et de recherches appliquées ou d'autres lignes directrices. Quand c'est possible, l'argumentation est étayée par divers exemples de recherche actuelle ou prévue.

Design Flows for the Shakwak Project

R. Kallio

Les crues maximales possibles dans le cadre du projet routier Shakwak, et de la reconstruction des routes Haines et de l'Alaska ont été évaluées par plusieurs méthodes. Les applications de ces méthodes sont présentées et certains résultats types sont donnés.

A Flexible Family of Stochastic Models

K.W. Hipel

La famille des modèles de Box-Jenkins et d'autres modèles stochastiques intimement reliés forme un groupe contribuant à résoudre de nombreux types de problèmes qui se posent en hydrologie. Un des modèles d'une importance particulière est le modèle d'intervention. Il peut servir à déterminer les effets anthropiques ou dus à des phénomènes naturels sur le niveau moyen d'une série chronologique, pour combler les observations qui y manquent et pour résoudre d'autres problèmes du même genre. Après en avoir fait la description, l'auteur explique comment, en se conformant aux étapes de l'identification, de l'estimation et de la vérification du diagnostic lors de l'établissement du modèle, on peut choisir le type de modèle d'intervention qui convient le mieux à une certaine série de données. Plusieurs exemples viennent confirmer l'importance du modèle appliqué aux données hydrologiques du pays.

Hydrologic Extreme Value Model and Risk Analysis Applied to Flood Plain Management

F. Ashkar, N. El-Jabi et J. Rousselle

Pour les auteurs, l'aménagement rationnel des plaines inondables peut se faire si on les divise en différentes zones à partir desquelles on peut surveiller leur mise en valeur et fixer, pour chacune d'elles, les primes d'assurance d'après les risques d'inondation. Ces risques peuvent être évalués d'après la relation entre la période de récurrence ou une crue donnée et le niveau des crues. Un modèle mathématique fondé sur la théorie de Gumbel appliquée au processus stochastique a été établi quand on a pu déterminer la fonction de distribution de ces valeurs extrêmes dans un intervalle de temps (0,t). On se sert du point de vue bayésien et des maximum probables afin de définir cette fonction pour une période de récurrence donnée quand on n'est pas sûr de ses paramètres, qui décrivent le modèle. Les auteurs comparent les risques que comporte l'évaluation de ces valeurs maximales pour différentes périodes.

Hydro Configuration Modelling System

D.W. Farley

Un programme de modélisation généralisée, dénommé HCMS (système de modélisation de configuration hydrologique), a été établi pour étudier des réseaux fluviaux à plusieurs réservoirs. Il s'applique particulièrement aux bassins qui produisent de l'énergie hydroélectrique ou qui pourraient le faire. Il sert à évaluer les projets concernant l'augmentation de l'emmagasinement dans les réservoirs pour la défense contre les crues; la capacité de nouvelles turbines; les prescriptions relatives à l'environnement définies sur la base de la limitation des niveaux des réservoirs ou de débits minimaux; les dérivations de courant; et l'augmentation de la capacité de débit des canaux. En comparant les résultats à différentes simulations de configurations et de plans d'exécution avec les situations qui se sont produites antérieurement, l'équipe de planificateurs peut facilement en évaluer les effets sur la défense contre les crues, sur la production d'énergie et l'environnement. Afin de faire avancer les travaux de modélisation, on a inclus dans le programme une routine réaliste de recherche de solutions du rendement optimal. Cette routine se fonde sur un sous-programme de programmation linéaire, tiré du produit-programme SL-MATH d'IBM. À la différence d'autres applications de programmation linéaire appliquées à des études des réseaux de ressources hydrauliques, la routine de ce modèle ne permet pas la parfaite analyse à postériori des données sur les séries chronologiques, et ne se limite pas à des périodes critiques de relevés pour se conformer aux exigences de dimensions communes aux autres modèles de rendement optimal.

A Procedure for Determining Flood Frequencies for Great Lakes Shoreline

D.W. Brown, R.J. Moulton et S.M.A. Moin

Les auteurs présentent des méthodes qu'on peut appliquer pour déterminer la fréquence des crues des Grands lacs. Le choix de la méthode dépend des données dont on dispose. Cette méthode est relativement simple près des stations de jaugeage pour lesquelles on possède des relevés du niveau des crues de pointe. Si l'on ne dispose pas de ces données, la méthode devient plus complexe car, pour établir la fréquence des crues, il faut calculer, à partir des données climatologiques, les montées de niveau dues au vent et combiner ces répartitions de fréquence avec celles du niveau moyen de l'eau du lac.

PART II

BACKGROUND PAPERS

PARTIE II

DOCUMENTS DE BASE

WORKING GROUP 1

Open Channel Hydraulic Modelling

GROUPE DE TRAVAIL N^{o} 1

Modélisation hydraulique des canaux ouverts

SIMPAK FOR RIVER BASIN STUDIES

by

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SIMPAK FOR RIVER BASIN STUDIES

INTRODUCTION

In several studies in the Pacific and Yukon Region, streamflow routing models have been used to determine the downstream effects of various reservoir or diversion projects. In the earlier studies, the SSARR program was adopted as one of the vehicles for routing studies in the region.

During the course of the Fraser River Upstream Storage Study, a streamflow routing model of the Fraser River was required for detailed daily regulation tests of the proposed reservoirs. The SSARR program was investigated as the basic tool for this job. It soon became apparent, however, that only a fraction of the capabilities of SSARR were required for this task. It would have been an advantage both in user's effort and in computer costs if the portions of that program which were not required could be removed and if additional specialized capabilities could be easily added to the program. An examination of the SSARR program showed that this could not be done without a great deal of reprogramming effort. It appeared the desired result could be achieved by simply using the routing methods of the SSARR program and building a separate program framework to do all the data handling. Thus, the simulation package (SIMPAK) for river modelling was created.

Over the past several years, the SIMPAK program has evolved much further with the development of several additional capabilities. It has been used in the Fraser River Upstream Storage Study to perform day-to-day streamflow forecasts for reservoir operation and for routing of reservoir holdouts to the damage center to estimate the amount of post-project flood peak reduction. It has also been used in the Columbia River Treaty flow computations to estimate the amount of flood peak reduction as a result of the construction of Treaty projects. Currently SIMPAK is being used as the basic tool for daily streamflow modelling of the Mackenzie River basin under work being done by the Mackenzie River Basin Committee. There are several other agencies in western Canada also using SIMPAK as basic tool in water resources management studies.

FEATURES OF THE SIMPAK PROGRAM

The SIMPAK program is a modular program which consists of a basic framework that reads the input instructions, sets up the logistics of the computer operations and controls the calling of the many separate modules to perform the computations. In developing the program the modular concept was adhered to as close as possible so that changes could easily be made to the program. Although it was originally developed to meet the needs of the Fraser River Upstream Storage Study, it has been written so as to be completely general in nature and can be applied to any river basin.

Each of the individual modules or subroutines in the SIMPAK program does some operations on a set of time-dependent data representing river flows, elevations, storage and change in storage values for each day of the simulation period. Each set of such data is handled in the main program as a "station" having a unique identification number. The various individual subroutines are then called from an input list of operation instructions. Each instruction gives the type of operation to be performed and parameter values to be used. Most of the functional information about the station is held within this single instruction.

The basic features of the program are described in the following sections:

(a) Input Data

There are several classes of data that are input to the program:

(1) The model control data that define the stations for the model and specify the operations that are to be performed at each node. This data constitutes "the model". All operations are specified on a standard format, making the coding and punching of the operating instructions a relatively easy matter. All the parameters associated with that operation are contained in the single instruction.

(2) Tables of relationships such as correlation curves, storage curves, backwater curves or other functions that are necessary in some operations. These are held on a separate data file that is set up externally to the program and that is automatically searched for the required tables.

(3) Hydrologic data such as records of discharges, elevations, etc. The program will read the required hydrologic data for the model from several different input sources such as card-image computer disc files, magnetic tape, and disc files holding unformatted data. On any of the data input units (i.e. cards, tape or disc) the data can be handled in any of three different data formats:

- WSC format, the format used by the Water Survey of Canada on their computer data tapes;
- SSARR format, the format used in the SSARR program for data input;
- "long records" format, made up of data records consisting of unformatted (i.e. machine code form) data.

Input instructions specify which sources are to be searched for the data. The program automatically searches all the data held in each of the specified sources to find the data required by the model.

Data may be in any station order and unused data may be included in the data files. A flag on each data record tells whether the data is in English or metric units. Mixtures of English and metric data can be handled automatically in any job.

(b) Model Time Control

The program works on a fixed time increment of one day except in SSARR reach routing where calculations are automatically made for shorter time periods under certain conditions. All input and output flows are mean daily discharges. Values of elevation and storage are computed as the end of day values.

The time boundaries for any run are input in the model control instructions. In its usual form the program handles up to 200 days in any cycle, however, this can easily be extended by re-dimensioning the program. All data is checked for the correct time period and only the data within that period is selected from the data files.

Any run of the program can be cycled to provide additional simulations for different time boundaries using the same input model. In such cases, the data applicable to the new cycle will be picked up automatically from the input data files.

(c) Organization of the Program

Each set of data within the program is indexed according to a unique station number. All operations of the program are then made by reference to these station numbers. Each operation is specified by an input operation card which tells the program what type of operation to carry out and specified the stations and parameters to use. Each operation takes the data held in one or more stations, uses that data in carrying out the specified operation, and stores the computed values of the operation under another specified station number. In this way the operations specified in the model control list also specify the configuration of the river.

For each station, there are four different types of data stored in the computer under index numbers as follows:

index 1 = discharges;
index 2 = elevations;
index 3 = storage;
index 4 = change in storage.

Thus, each station has an array for each of these four types of data. They are not necessarily all filled in with meaningful data, depending upon the operations that are carried out using that station.

There also may be tables associated with any station. If the operation being carried out requires characteristic data or control data in addition to the parameters specified on the control list, the program looks for a table having the station number as its identification.

(d) Operation of SIMPAK

There are about 25 operations that can be carried out by SIMPAK. The SIMPAK User's Manual, available from the IWD Pacific and Yukon Regional Office, shows the specifications for the operation control cards. Some of the frequently used operations are:

(1) "LAKE" ... routes flows through a free-flow lake, using the Modified Puls Method of lake routing;

(2) "RESEV" ... routes flows through a reservoir based on a control record consisting of specified values for any one of the four data types (discharge, elevation, storage or change in storage), with constraints for upper and lower reservoir limits, minimum flows and discharge capacity;

(3) "REACH" ... routes flows through a river channel reach using the SSARR method of channel routing;

(4) "WAVE" ... routes river flows or reservoir holdouts to a downstream point in the river using a polynomial method of routing (similar to a unit hydrograph);

(5) "ADD" ... sums the data from two or more stations;

(6) "SUBT" ... subtracts the data for one station from another;

(7) "TRSF" ... multiplies data for a station by any specified constant; this can be used to extend records on the basis of input daily ratios between stations' data;

(8) "MEAN" ... computes monthly means and totals of data values;

(9) "LOOK" ... computes discharge at a station from specified water surface elevations or vice-versa;

(10) "SET" ... sets data for a station to a constant throughout the period of the model;

(11) "BLOO" ... does a stage/discharge/control table look-up under backwater conditions; the operation must be located within a backwater string (see operation BWST below);

(12) "BLAK" ... does a lake routing computation under backwater conditions. It must be located within a backwater string (see operation BWST below);

(13) "BWST" and "BWEN" ... allows some of the above operations to be carried out under backwater conditions where iterative procedures are necessary.

In addition to these operations, there is a dummy operation "USER" that allows a person to program his own routine to fit into the SIMPAK structure. Any other program may be called by the SIMPAK model control list by identifying the program with the name "USER" and concatenating it to the SIMPAK program. This is a very handy feature that allows a great deal of flexibility in using the program.

(e) Data Output

SIMPAK outputs data in several forms. The most common is a listing of the discharges for each station showing on each page one month of mean daily discharges for eight stations. Examples of such output are shown in Figure 4.

The complete set of data for a station, i.e. discharge, elevation, storage and change in storage, can be printed if specified by the input model instructions. An example of such output is shown in Figure 3.

The program will also output data to computer files in either the SSARR format or the long record format. An associated program is available to convert either of these to the WSC format.

Two different types of plotting can be accomplished by the program. The most convenient type is the printer-plot, in which the printer output sheet is scaled and a hydrograph is "plotted" using printer symbols. This method provides a fast and inexpensive plot of any specified time-dependent data. Several stations can be plotted together to give a clear comparison of the data at those stations. The second type of plot, having the same features and format as the printer-plot, is the line-plot produced on a plotter such as the Calcomp.

All data output can be in either English or metric units, depending on the specifications in the model control instructions.

COMPARISON OF SIMPAK AND SSARR

The SSARR program is an internationally well known computer package for streamflow routing and reservoir regulation. The program performs three basic tasks of watershed modelling, river system modelling and reservoir regulation modelling. Although SIMPAK is not as comprehensive as SSARR it has many advantages over SSARR. A comparison between the two programs is necessary in order to assist users in selecting the best tool for their study purpose:

		SIMPAK	SSARR
Simplicity	-	Programming features possess inherent simplicity. Modular components for all operation functions.	One large complex program with inflexible programming features and complex program logic.

		SIMPAK	SSARR
Flexibility	-	Additional operations can be easily added or deleted.	Has no ready facility to make further refinements.
Data Handling	-	Is capable of accepting various data formats and interfacing directly with data base management systems.	Is restricted to a single input format.
Input Verification	-	Easier to verify since operation command and "station links" are visible on one card.	Difficult to verify.
Unit Conversion	-	Completely flexible.	Has to use either English or metric units throughout the run.
Interfacing with Other Models	-	Has a programmed "USER" routine for specific needs.	Maintains no programmable entry point.
Watershed Model	-	No present capability but can be interfaced easily.	Readily available capability.
Variable Time	-	Fixed daily time period for input and output, with a computational time interval which varies internally within the "REACH" routing routine.	Completely variable time operations for all functions.

In addition, it has been estimated that the time required to set-up the model and cost to run the model for SIMPAK is about 30% less than that for SSARR.

SIMPAK APPLICATIONS

(a) Fraser River Upstream Storage Study

The nature of this study was to evaluate the flood control effectiveness of a system of proposed reservoirs. The flood control can be measured by the reductions of flood damages at various magnitudes of floods. To measure the flood peak reduction, a simulation study is required in order to simulate the operation of the proposed reservoirs for flood control under conditions similar to that under the real-time condition. A real-time flood control operation usually involves the forecasts of freshet runoff volume, target regulated levels and expected daily inflow.

The proposed reservoir sites for the Fraser Study were located far upstream from the major flood damage area in the Lower Fraser Valley and the effect of changes in flows due to reservoirs would not reach the Lower Valley until three to four days later. This time lag makes the use of daily streamflow forecasts necessary in day-to-day scheduling of reservoir holdouts. An empirical method of simulating real-time forecasts was derived based only on available streamflow information up to the current day. The procedure was integrated with SIMPAK through the USER operation. The procedure involved the selection of a set of key gauging stations and local inflows, producing one through four-day forecasts for each of these stations. Examination of plots of the hydrographs at the key stations indicated that the required forecasts could be made simply by extrapolating the individual upstream hydrographs based on slope and curvature. These individual forecasts are then added and routed to the flood control centre. The forecast algorithm was derived empirically and varies with each upstream station. The underlying principle in creating the algorithm was that forecast should always simulate the worst flow condition that would occur during the forecast period. Using past flow information, the forecasted flow at flood damage centres is then calculated by routing and summing the recorded and forecasted flow from each of the selected upstream stations. The forecast method described herein was developed through a number of tests in which the parameters used were calibrated to give a workable forecast for use in flood control regulations.

A flood control simulation model was developed and operated for the Fraser River Basin on a day-to-day basis. Holdouts for flood control purposes were scheduled each day at the flood control centre and apportioned to upstream reservoirs according to relative emptiness and refill probability of each individual reservoir. The simulation procedure accounts for the simple time lag between reservoirs and flood control centre. For example, a scheduled holdout in the amount of 10,000 cfs at flood control centre for day 10 is apportioned to a reservoir considered having 4-day time lag, the actual storage at the reservoir is performed on day 6 in the amount of 10,000 cfs, if there is enough inflow. The day-to-day reservoir holdouts are loaded onto a computer file and retrieved subsequently by SIMPAK which performs a detailed routing of the holdouts downstream to flood control centre. The regulated flows at flood control centre are then computed by subtracting routed holdouts from unregulated flows as shown in Figure 6 for the Fraser River at Mission.

(b) Columbia River Pre-project Flow Computation

As a result of the signing of the Columbia River Treaty in 1961, a Permanent Engineering Board was established and one of the duties of the Board would be to "make reports to Canada and the United States of America at least once a year of the results being achieved under the treaty ....." One of the items in the annual report by the Board is to show the flood control effects by the treaty reservoirs, namely Duncan, Arrow, Mica, and Libby. The effects can best be measured by flood peak reduction through the comparison of observed flows and pre-project flows that would have occurred if the dams had not been built. Therefore, the computation of the pre-project flows become necessary as part of the tasks assigned to the Board.

The SSARR program was originally adopted as the tool for pre-project flow computation. When all projects were completed in 1973, the model had become very large and cumbersome. Since SIMPAK could do the job more efficiently, it was decided in 1975 to convert from the SSARR to the SIMPAK model. No difficulties were encountered during the process of conversion and the results obtained were similar to SSARR results. The flow chart and computer card image of the SIMPAK model for Columbia River pre-project flow computation are shown in Figure 1 and 2 respectively. Some of the useful features in the model are given below:

1. "Back routing" with zero inflows through reservoir to compute "actual" inflows i.e. stations (2), (26), and (29).
2. "Local inflow computation" by routing observed flows through the reach under consideration, i.e. (30) to (33), then obtain (35) from (33) and 34).
3. "Lake routing" under normal condition, i.e. stations (8) and (17).
4. "Backwater lake routing" through a string of lakes under backwater conditions, i.e. stations (42) and (43).
5. "Pre-project flow computation" by routing pre-project flow from upstream station to downstream station and adding local inflow to it, i.e. (44) to (45) then, compute (46) from (45) and (25).

The result of 1976 flood control regulation at the town of Trail, which is located some 10 miles north of international border is given in Figure 5. The hydrographs show that without regulation by the Treaty storage projects, the 1976 flood peak would have reached 236,000 cfs and would have caused minor flooding at Trail.

(c) Mackenzie River Studies

As part of the Mackenzie River Basin Study presently being carried out under the Canada Water Act, a daily routing model of the major tributaries of the Mackenzie River is being prepared by the Inland Waters Directorate. This model will provide a tool to analyse changes in the hydrologic regime of the river due to proposed dams and reservoirs on the river system. The SIMPAK program is being used to develop and run this model.

For the development of the model, the basin was divided into three sub-basins (Peace-Slave-Mackenzie System, Athabasca-Lake Athabasca, Liard River) and each was assigned to that office of Inland Waters Directorate having the relevant data and experience. For functional purposes and for the actual operation, the model will consist of several sections. Each section models a major tributary or a section of the Mackenzie River between these tributaries. These sections will be pieced together as necessary to route flows downstream through the system.

Work is still proceeding on the development of this model. This is a standard application of a routing model for which SIMPAK is well suited. It will be very similar to the previously described application on the Columbia River. Recorded flows will be used to determine the local contributions along the river. Then project flows will be substituted for recorded flows and these will be routed through the model, picking up the local contributions along the way. The model will be used to study the effects of proposed developments and to estimate natural conditions where there are existing projects.

CONCLUSION AND RECOMMENDATIONS

The SIMPAK program has been developed and used by the Vancouver office of Inland Waters Directorate for several river basin studies. It has been used to perform river routing and reservoir regulation, critical flood flow forecasting and other operations involving manipulation of streamflow data. The critical flood flow forecasting was developed specifically for day-to-day reservoir flood control operation. The technique derived for the forecasts is a simplified approach yet provides a workable and useful tool for reservoir operation.

Users are reminded that SIMPAK should be distinguished from those programs for backwater computations which produce river profiles or rating curves. Users should also be reminded that no decision-making algorithm was built into the SIMPAK reservoir regulation routine although an algorithm can be programmed separately and interfaced with SIMPAK via the USER routine. It should be pointed out that SIMPAK has no watershed modelling capacity and it is recommended that existing watershed models should be investigated and an acceptable one be interfaced with SIMPAK whenever necessary.

Although SIMPAK is not as comprehensive as SSARR, it has advantages over SSARR for time and cost savings in setting up and running the model. SIMPAK is especially convenient in data handling because it is capable of accepting various data formats including WSC format, and it can easily be modified to suit any other formats. It is recommended that SIMPAK be used over SSARR whenever daily routing study is required, because of its simplicity and flexibility.

The SIMPAK program has been well accepted in Canada and is presently being used by several agencies in Western Canada for various studies. It is recommended that SIMPAK be considered as a basic tool for any water resources study where streamflow simulation or the manipulation of streamflow data is required.

Figure 1.

COLUMBIA SIMPAK MODEL

PRE-PROJECT FLOW COMPUTATION

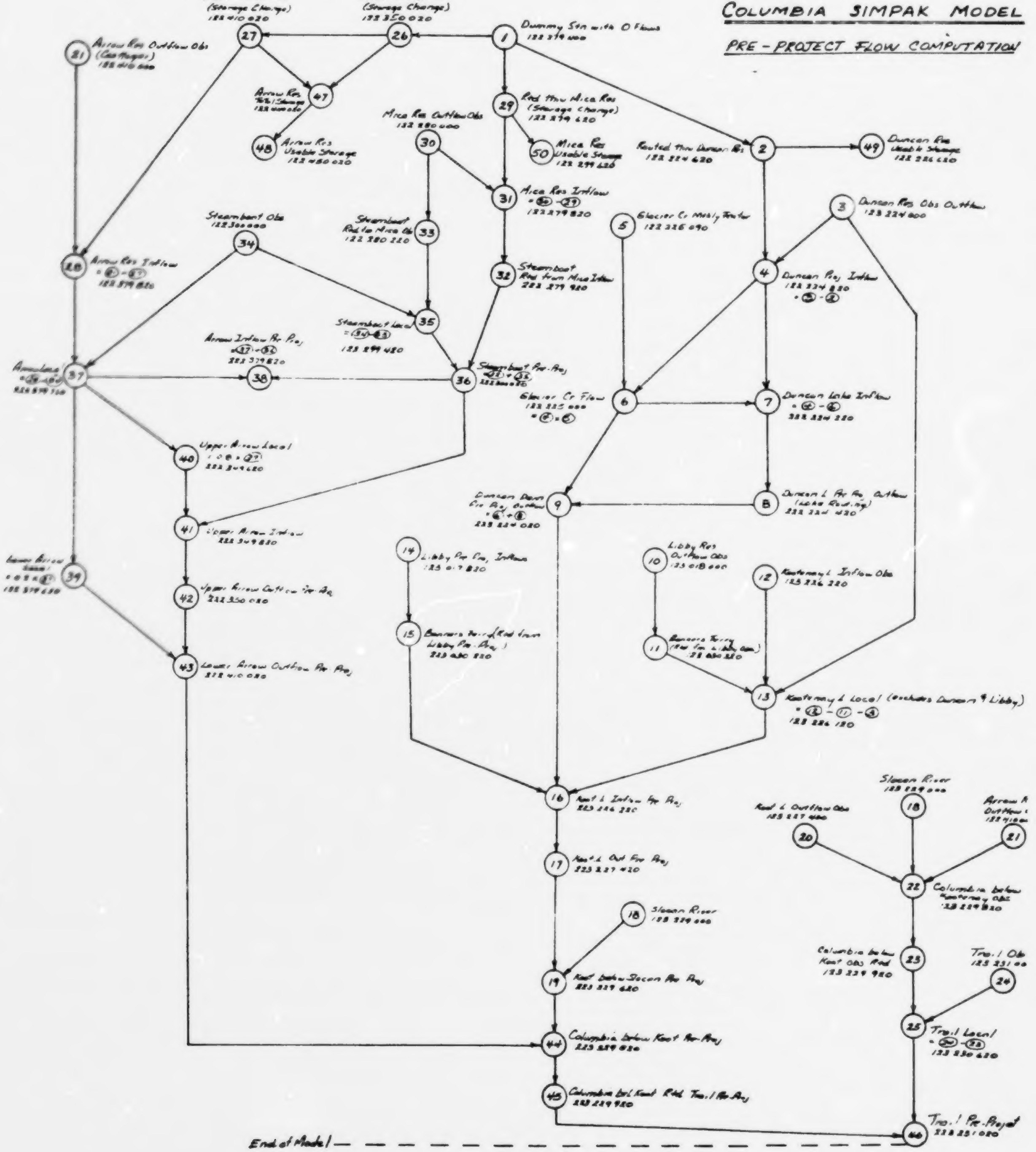
1 Dummy Stn with O Flows 122 279 400
26 Rtd thru U A L (Storage Change) 122 350 020
27 Rtd thru L A L (Storage Change) 122 410 020
21 Arrow Res Outflow Obs (Castlegar) 122 410 000
47 Arrow Res Total Storage 122 400 020
48 Arrow Res Usable Storage 122 480 020
29 Rtd thru Mica Res (Storage Change) 122 279 620
50 Mica Res Usable Storage 122 299 620
30 Mica Res Outflow Obs 122 280 000
31 Mica Res Inflow = 30 - 29 122 279 820
2 Routed thru Duncan Res 122 224 620
49 Duncan Res Usable Storage 122 226 620
3 Duncan Res Obs Outflow 123 224 000
5 Glacier Cr Mthly Factor 122 226 090
4 Duncan Proj Inflow 122 224 820 = 3 - 2
34 Steamboat Obs 122 300 000
33 Steamboat Rtd to Mica Obs 122 280 220
32 Steamboat Rtd from Mica Inflow 222 279 920
28 Arrow Res Inflow = 21 - 27 122 279 820
35 Steamboat Local = 34 - 33 123 299 420
36 Steamboat Pre-Proj = 35 + 32 222 300 020
37 Arrow Local = 28 - 34 222 279 720
38 Arrow Inflow Pre Proj = 37 + 36 222 379 820
6 Glacier Cr Flow 122 225 000 = 4 × 5
7 Duncan Lake Inflow = 4 - 6 222 224 220
8 Duncan L Pre Proj Outflow (Lake Routing) 222 224 420
9 Duncan Dam Pre Proj Outflow = 6 + 8 223 224 020
40 Upper Arrow Local = 0.8 × 37 222 349 620
41 Upper Arrow Inflow 222 349 820
39 Lower Arrow Local = 0.2 × 37 122 379 620
42 Upper Arrow Outflow Pre Proj 222 350 020
43 Lower Arrow Outflow Pre Proj 222 410 020
14 Libby Pre Proj Inflow 123 017 820
15 Bonners Ferry (Rtd from Libby Pre Proj) 223 030 220
10 Libby Res Outflow Obs 123 018 000
11 Bonners Ferry (Rtd fm Libby Obs) 123 030 220
12 Kootenay L Inflow Obs 123 226 220
13 Kootenay L Local (excludes Duncan & Libby) = 12 - 11 - 3 123 226 120
16 Koot L Inflow Pre Proj 223 226 220
17 Koot L Out Pre Proj 223 227 420
18 Slocan River 123 229 000
19 Koot below Slocan Pre Proj 223 229 620
44 Columbia below Koot Pre Proj 223 229 820
45 Columbia bel Koot Rtd Trail Pre Proj 223 229 920
20 Koot L Outflow Obs 123 227 400
18 Slocan River 123 229 000
21 Arrow R Outflow 122 410 00
22 Columbia below Kootenay Obs 123 229 820
23 Columbia below Koot Obs Rtd 123 229 920
24 Trail Obs 123 231 00
25 Trail Local = 24 - 23 123 230 620
46 Trail Pre-Project 223 231 020
End of Model

Figure 2 - Computer Card Image for the Columbia SIMPAK Model

```
 1   COLUMBIA SEPT/75-SEPT/76 PRE-PROJECT COMPUTATIONS
 2        122279400         1        DUMMY STN. WITH 0 FLOWS
 3        122224620         9 1      STORAGE CHANGE RTD THRU DUNCAN RES.(FEB/73 TABLE)
 4        123224000         1        DUNCAN RESERVOIR OBS.
 5        122224820         1        DUNCAN PROJECT INFLOW
 6        122225090         1        GLACIER CR. MONTHLY FACTOR
 7        122225000         1        GLACIER CR. FLOW
 8        222224220         1        DUNCAN LAKE INFLOW
 9        222224420         9        DUNCAN LAKE PRE-PROJECT OUTFLOW (APR/68 TABLE)
10        223224020         1        DUNCAN DAM PRE-PROJECT
11        123018000         1        LIBBY RES. OUTFLOW OBS.
12        123030220         1        BONNERS FERRY(RTD FROM LIBBY OBS.)
13        123226220         1        KOOTENAY LAKE INFLOW OBS.
14        123226120         1        KOOT. L. LOCAL(EXCLUDES DUNCAN AND LIBBY)
15        123017820         1        LIBBY PRE-PROJECT INFLOWS
16        223030220         1        BONNERS FERRY(RTD FROM LIBBY PRE-P.)
17        223226220         1        KOOTENAY LAKE INFLOW PRE-PROJECT
18        223227420         9        KOOTENAY L. OUT PRE-P.(DISH.1941;ST.MAR24/69)
19        123229000         1        SLOCAN R.
20        223229620         1        KOOTENAY BELOW SLOCAN PRE-PROJECT
21        123227400         1        KOOTENAY LAKE OUTFLOW OBS.
22        122410000         1        ARROW RESERVOIR OUTFLOW OBS.
23        123229820         1        COLUMBIA BELOW KOOTENAY OBS.
24        123229920         1        COLUMBIA BELOW KOOTENAY OBS. RTD
25        123231000         1        TRAIL OBS.
26        123230620         1        TRAIL LOCAL
27        122350020         9 1      STORAGE CHANGE RTD THRU U.A.L. (FEB/73 TABLE)
28        122410020         9 1      STORAGE CHANGE RTD THRU L.A.L. (JAN/74 TABLE)
29        122379820         1        ARROW RESERVOIR INFLOW
30        122279620         9 1      STORAGE CHANGE RTD THRU MICA RES.(MAR/74 TABLE)
31        122280000         1        MICA RES. OUTFLOW OBS.
32        122279820         1        MICA RES. INFLOW
33        222279920         1        STEAMBOAT, RTD FROM MICA INFLOW
34        122280220         1        STEAMBOAT, RTD FROM MICA OBS.
35        122300000         1        STEAMBOAT OBS.
36        122299420         1        STEAMBOAT LOCAL
37        222300020         1        STEAMBOAT, PRE-PROJECT
38        222379720         1        ARROW LOCAL
39        222379820         1        ARROW INFLOW PRE-PROJECT
40        122379620         1        LOWER ARROW LOCAL
41        222349620         1        UPPER ARROW LOCAL
42        222349820         1        UPPER ARROW INFLOW
43        222350020         9        UPPER ARROW OUTFLOW PRE-PROJECT (FEB/73 TABLE)
44        222410020         9        LOWER ARROW OUTFLOW PRE-PROJECT (JAN/74 TABLE)
45        223229820         1        COLUMBIA BELOW KOOT. PRE-PROJECT
46        223229920         1        COLUMBIA BELOW KOOT. RTD TRAIL PRE-PROJECT
47        223231020         1        TRAIL PRE-PROJECT
48        122440020                  ARROW RES.  TOTAL STORAGE
49        122450020                  ARROW RES. USABLE STORAGE
50        122226620                  DUNCAN RES. USABLE STORAGE
51        122299620                  MICA RES. USABLE STORAGE
52   01 09 1975  30 09 1976           1   1                   1
53   SET   122279400                                 1        0
54   RESE  122224620 122279400                       2 9 9  1435759
55   SUBT  122224820 123224000 122224620             1 1 1
56   MULT  122225000 122224820 122225090             1 1 1
57   SUBT  222224220 122224820 122225000             1 1 1
58   LAKE  222224420 222224220                               4674
59   ADD   223224020 222224420 122225000             1
60   PEAC  123030220 123018000                       3        0.2      40.0
```

Figure 2 - (continued)

```
   61     SUBT  123226120 123226220 123030220 123224000 1 1 1
   62     REAC  223030220 123017820                     3              0.2       40.0
   63     ADD   223226220 223030220 223224020 123226120 1
   64     LAKE  223227420 223226220                                  26150
   65     ADD   223229620 223227420 123229000           1
   66     ADD   123229820 123227400 122410000 123229000 1
   67     REAC  123229920 123229820                     2              0.2       10.0
   68     SUBT  123230620 123231000 123229920           1 1 1
   69     RESE  122350020 122279400                     2 9 9  4662159
   70     RESE  122410020 122350020                     2 9 9  3140143
   71     SUBT  122379820 122410000 122410020           1 1 1
   72     RESE  122279620 122279400                     2 9 9 13639900
   73     SUBT  122279820 122280000 122279620           1 1 1
   74     REAC  222279920 122279820                     6              0.2       27.0
   75     REAC  122280220 122280000                     6              0.2       27.0
   76     SUBT  122299420 122300000 122280220           1 1 1
   77     ADD   222300020 122299420 222279920           1
   78     SUBT  222379720 122379820 122300000           1 1 1
   79     ADD   222379820 222300020 222379720           1
   80     TRSF  122379620 222379720                     1 1            0.2
   81     TRSF  222349620 222379720                     1 1            0.8
   82     ADD   222349820 222349620 222300020           1
   83     RWST
   84     BLAK  222350020 222349820           222410020  2           53500    1389.68
   85     BLAK  222410020 222350020 122379620 223229620  1           56400    1382.23
   85.5   RWEN
   86     ADD   223229820 222410020 223229620           1
   87     REAC  223229920 223229820                     2              0.2       10.0
   88     ADD   223231020 223229920 123230620           1
   89     ADD   122440020 122350020 122410020           3
   90     SUBT  122226620 122224620                     3 3                                  32400
   91     SUBT  122450020 122440020                     3 3                                1237000
   92     SUBT  122299620 122279620                     3 3                                8075200
  100     SARP                                          2               20         10
  101      122280000         1         0          0   10000
  102      122279820         1         1          0   10000
  103     SARP                                          3               20         10
  104      122410000         1         0          0   20000
  105      222410020         1         1          0   20000
  106      222379820         1         2          0   20000
  107     SARP                                          2               20         10
  108      123018000         1         0          0   10000
  109      123017820         1         1          0   10000
  110     SARP                                          2               20         10
  111      123224000         1         0          0    2500
  112      223224020         1         1          0    2500
  113     SARP                                          2               20         10
  114      123231000         1         0          0   25000
  115      223231020         1         1          0   25000
  116     SARP                                          1               30         10
  117      122450020         3         0   -1000000 1000000
  118     SARP                                          1               30         10
  119      122226620         3         0    -500000  500000
  120     SARP                                          1               30         10
  121      122299620         3         0    3000000 1000000
 END OF FILE
```

Figure 3 - Example of Complete Printout for One Station

```
                     COLUMBIA SEPT/75-SEPT/76 PRE-PROJECT COMPUTATIONS

                              STATION  222410020

                    LOWER ARROW OUTFLOW PRE-PROJECT (JAN/74 TABLE)

        DATE              OUTPUT         ELEVATION      STORAGE     DELTA STORAGE
                          (CFS)           (FEET)       (ACRE FT)     (CFS DAYS)


   1 JULY 1976        107529.25          1392.00       1158359.       15148.06
   2 JULY 1976        113900.44          1392.76       1187864.       14875.56
   3 JULY 1976        118905.00          1393.16       1203433.        7849.50
   4 JULY 1976        120922.12          1393.19       1204761.         669.62
   5 JULY 1976        121041.62          1393.14       1202954.        -910.62
   6 JULY 1976        121705.37          1393.35       1210826.        3968.81
   7 JULY 1976        124669.06          1393.88       1231852.       10600.62
   8 JULY 1976        130251.00          1394.68       1263076.       15742.18
   9 JULY 1976        137015.44          1395.46       1293432.       15304.87
  10 JULY 1976        142783.12          1395.99       1314028.       10384.12
  11 JULY 1976        145885.75          1396.12       1319268.        2642.19
  12 JULY 1976        145959.69          1395.95       1312828.       -3246.62
  13 JULY 1976        144146.94          1395.67       1301741.       -5589.37
  14 JULY 1976        142036.75          1395.41       1291507.       -5159.31
  15 JULY 1976        140585.31          1395.24       1284928.       -3316.62
  16 JULY 1976        140050.62          1395.21       1283632.        -653.37
  17 JULY 1976        140832.06          1395.34       1288715.        2562.87
  18 JULY 1976        142617.19          1395.51       1295284.        3312.06
  19 JULY 1976        144352.00          1395.63       1300200.        2478.56
  20 JULY 1976        145559.50          1395.69       1302520.        1170.00
  21 JULY 1976        145995.81          1395.62       1299842.       -1349.69
  22 JULY 1976        144393.44          1395.21       1283810.       -8082.50
  23 JULY 1976        141153.62          1394.77       1266706.       -8623.25
  24 JULY 1976        138208.37          1394.41       1252472.       -7176.31
  25 JULY 1976        135815.69          1394.11       1240541.       -6014.93
  26 JULY 1976        134770.87          1394.04       1237968.       -1297.06
  27 JULY 1976        134724.50          1393.96       1234660.       -1667.56
  28 JULY 1976        133385.31          1393.59       1220268.       -7255.75
  29 JULY 1976        130116.62          1393.07       1200026.      -10205.06
  30 JULY 1976        126148.12          1392.56       1180097.      -10047.25
  31 JULY 1976        122350.44          1392.07       1160980.       -9638.06
```

Figure 4 - Example of Eight Station Discharge Printout

```
                         COLUMBIA SEPT/75-SEPT/76 PRE-PROJECT COMPUTATIONS

                                        OUTFLOW (CFS)

                    122379620   LOWER ARROW LOCAL
                    222349620   UPPER ARROW LOCAL
                    222349820   UPPER ARROW INFLOW
                    223229820   COLUMBIA BELOW KOOT. PRE-PROJECT
                    223229920   COLUMBIA BELOW KOOT. RTD TRIAL PRE-PROJECT
                    223231020   TRIAL PRE-PROJECT

                  STATION     STATION     STATION     STATION     STATION     STATION    STATION  STATION
     DATE        122379620   222349620   222349820   223229820   223229920   223231020

 1 JULY 1976      9623.48    33493.95   143485.81   197387.19   197009.87   198059.12
 2 JULY 1976      7896.89    31595.55   137371.87   204032.06   203548.25   203905.37
 3 JULY 1976      6854.96    27419.45   118627.69   206710.81   208372.37   208008.87
 4 JULY 1976      6586.06    27944.37   110900.50   210165.69   210060.12   210008.25
 5 JULY 1976      7191.46    28763.83   110883.12   210310.00   210298.75   209933.87
 6 JULY 1976      8712.87    34851.50   128993.94   210572.31   210923.75   210289.62
 7 JULY 1976      9837.87    39351.50   149397.37   215127.94   214829.25   213520.87
 8 JULY 1976     10283.65    41134.55   163434.56   222127.69   221626.37   220482.25
 9 JULY 1976      9390.56    37562.25   163767.44   224873.25   229321.87   227799.12
10 JULY 1976      9033.41    36133.64   153263.94   235426.25   235034.00   234541.62
11 JULY 1976      8201.29    32805.14   139041.50   236167.12   237973.87   236283.37
12 JULY 1976      7775.50    31102.00   127662.75   238057.00   238064.12   236339.50
13 JULY 1976      7354.30    29417.20   123275.12   235642.25   235811.00   234245.37
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21 JULY 1976      6666.39    25665.55   137629.87   228224.37   228271.62   227054.37
22 JULY 1976      2459.74     9838.95   117524.87   225001.44   225228.75   231191.75
23 JULY 1976      4939.50    15753.00   109581.81   219944.37   220213.75   220398.37
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25 JULY 1976      6171.00    24604.00   129131.62   210745.25   211056.12   207701.37
26 JULY 1976      4858.98    19435.95   132206.12   207971.44   208169.62   204251.75
27 JULY 1976      4761.91    19047.65   117710.31   206217.87   206343.12   203517.37
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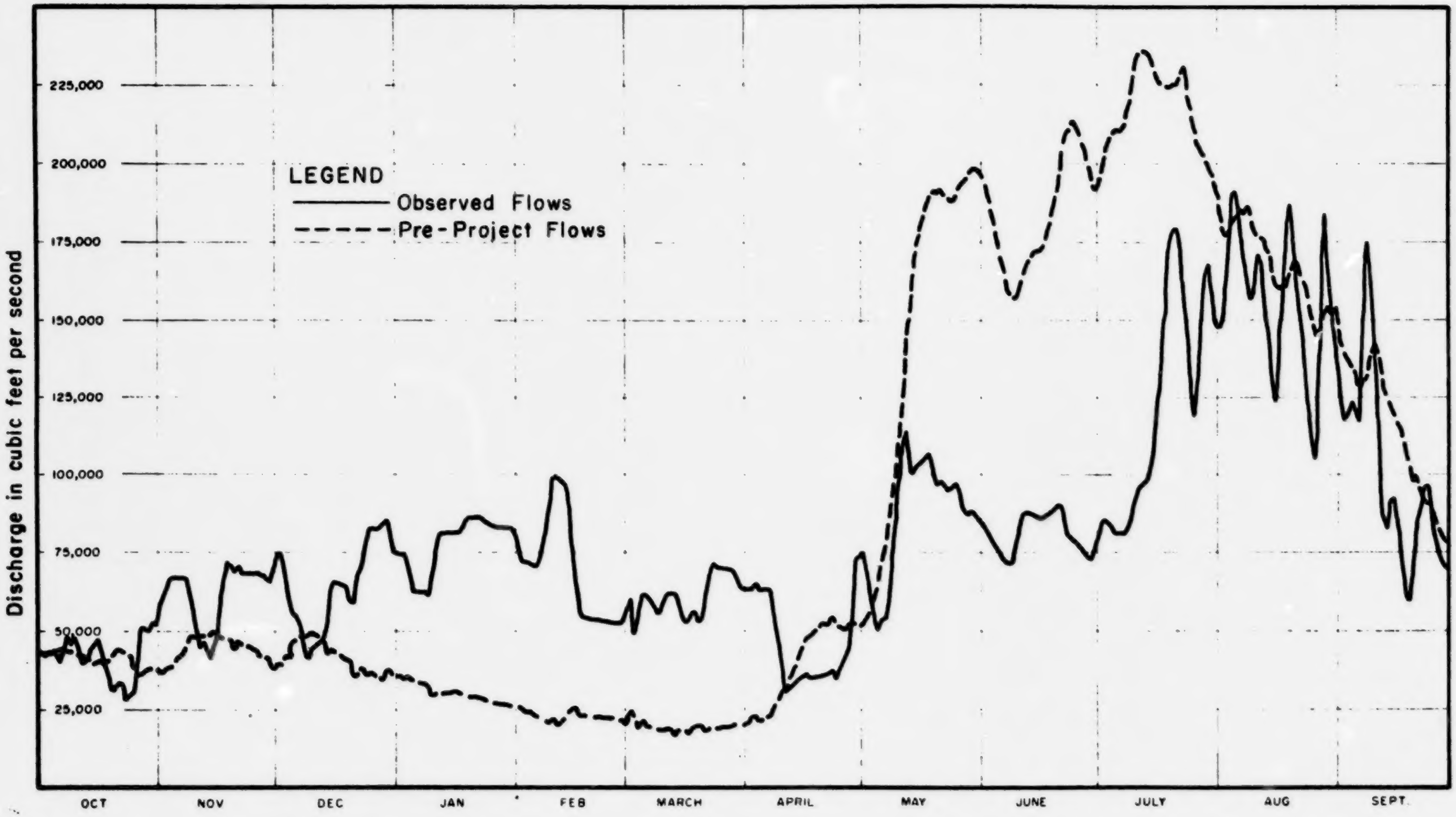

Figure 5 – COLUMBIA RIVER AT BIRCHBANK *(near Trail)*

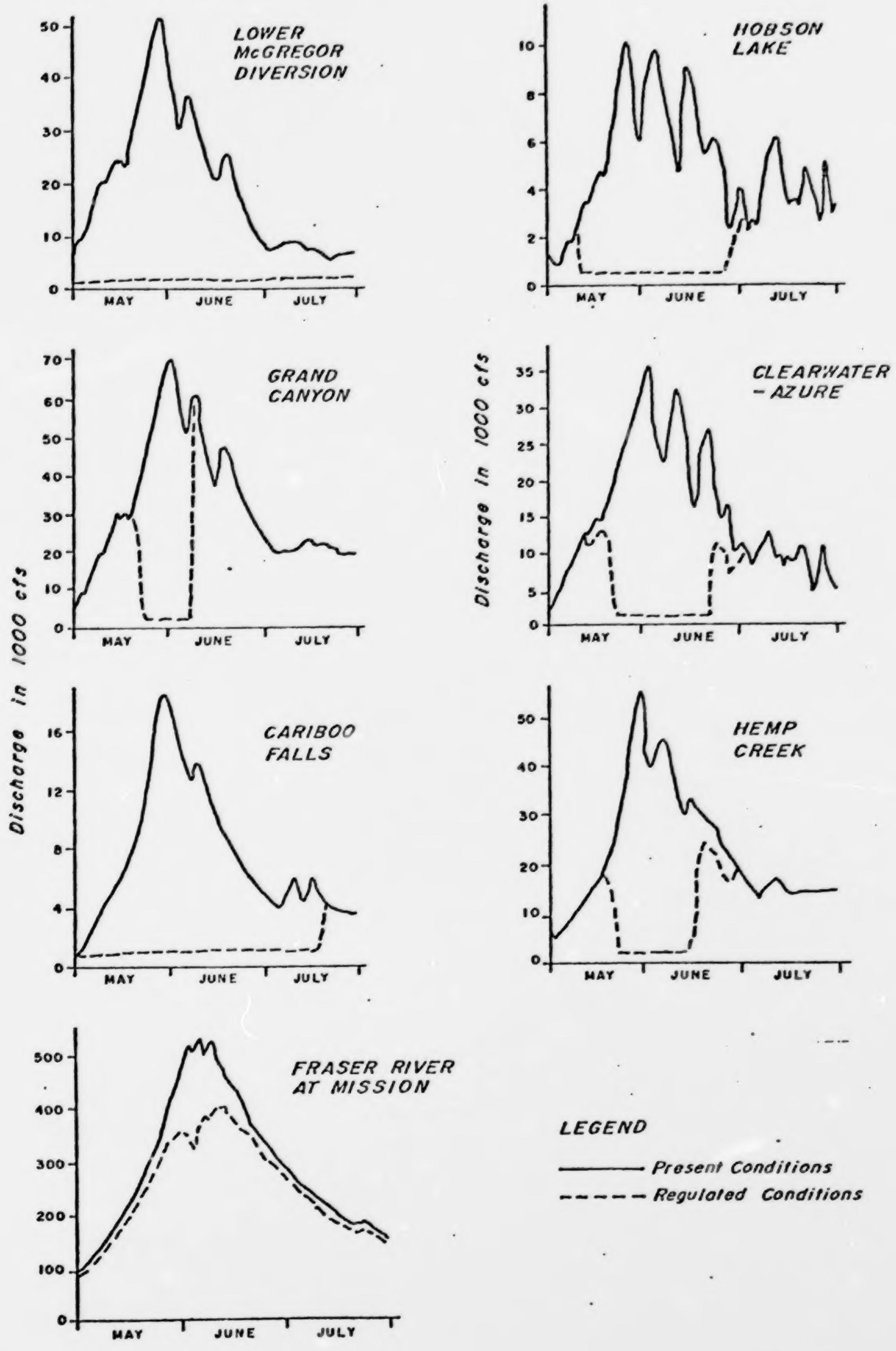

Figure 6 – 1948 Freshet Regulated by All System E Projects

PROGRESS REPORT

DEVELOPMENT OF A REAL-TIME OPERATIONAL MODEL FOR THE OTTAWA RIVER

prepared by:

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Introduction

Flooding has always been part of the way of life in the Montreal Region but, until recently, it generally caused only minor damage. In the 1960's, many seasonal residences were converted into permanent homes and, unfortunately, this development was followed by the occurrence of several large floods.

This led to considerable pressure on governments to alleviate the problem and in May, 1974, a Canada-Quebec agreement was signed that initiated studies on flooding and low water problems in the Montreal Region. These studies took more than two years to complete and culminated in a final report in October, 1976. In that report, it was recognized that any potential for reducing flooding in the Montreal Region lay in optimizing the operation of the many reservoirs in the Ottawa River basin.

As a result of the recommendations in the report, the Ottawa River Regulation Planning Committee was formed in February, 1977, to carry out the following terms of reference:

1 - Recommend criteria for regulation of the Ottawa River, taking into account hydro-power production, flood protection, navigation, low water problems, water quality needs and recreation. In this work the committee will take into account the results of studies of possible additions to storage and improvements in operation of existing reservoirs.

2 - Refine and recommend an operational application of a forecasting model.

3 - Recommend on development of a flood warning system.

4 - Recommend an effective liaison mechanism with St. Lawrence River regulation.

5 - Recommend on the nature and funding of a continuing Ottawa River operating committee.

The Committee formed Subcommittee 1 to carry out the first term and a Working Group was then formed to develop a mathematical model of the regulation of the Ottawa River.

The first task was to study several models that have been developed or adapted for various purposes on the Ottawa River basin.

HCMS was a regulation model developed by Subcommittee 4 of the Committee on Flow Regulation - Montreal Region. Its objective was to maximize potential energy while satisfying primary power and this was unacceptable to some of the agencies involved in the present study.

At the same time, the SSARR model was applied to the Ottawa River with a view to developing a flood forecasting system. Both the watershed and routing parts of the model were calibrated for this purpose but no operating policies were assumed or recommended. Declarations of outflows or reservoir levels on a daily basis were used as input to the model.

Subsequent to the work of the Committee on Flow Regulation - Montreal Region, another model called HCOMS was developed. This was a short-term model to be used in conjunction with natural inflow forecasts that would recommend operating policies. Again, the objective was to maximize potential energy while respecting various constraints throughout the system.

Subcommittee 1 and the Working Group studied these models in light of the terms of reference and, after some consideration, a proposal for a new approach was presented and accepted by the Subcommittee and the Committee.

Purpose

Two main purposes were identified for a modelling system of the Ottawa River basin. The first is to study the possibilities of modifying the operation of the reservoirs in order to reduce flooding in the Montreal Region (which can be expressed in terms of peak flow at the Carillon generating station). The second is to measure the effect of such modifications on the production of power in the basin.

It should be noted at this point that typically there are two flood peaks in the Ottawa River at Carillon. The first is usually the result of early snowmelt in the southern part of the basin while the second, normally larger peak, is from the northern watersheds. As the southern part is virtually uncontrolled, no modified operation will significantly affect the first peak. Also it has been shown that there are no possibilities for new storage in that region that would reduce the flood.

Approach

Up to now, the Ottawa River basin has been operated by experienced people in order to maximize power production while respecting a great number of local constraints on flooding, navigation, recreation and low flows.

The basic approach of this study, therefore, is to take the two main components (power production and flooding in the Montreal Region) into account by considering a composite objective function. The first objective is to maximize the worth of power generation as:

$$F = \max[C_1 \text{ (worth of power generation)}]$$

and the second is to minimize flood damages in the Montreal region as:

$$F = \min[C_2 \text{ (flood damages)}]$$

In this way, both objectives are expressed in common terms (dollars) and the two coefficients can be adjusted to give a range of alternatives. If $C_1 = 1$ and $C_2 = 0$, the objective is to maximize the worth of power without considering flood damages. Conversely, if $C_1 = 0$ and $C_2 = 1$, the objective is to minimize damages while ignoring the effect on power. There are of course an infinite number of alternatives between the two extremes which satisfy $C_1 + C_2 = 1$, and it is hoped that a solution can be found that represents an acceptable balance between power production and flooding.

Model Philosophy

In order to optimize the production of power in the basin, it was recognized that a model with a time horizon of one year is needed due to the annual pattern of energy requirements. It is also known that, particularly in the freshet period, operating decisions may be made on a daily basis depending upon current and forecasted conditions. Also, to enable reasonable decisions to be made on balancing the production of power and flooding, a model encompassing the entire freshet period is desirable. Thus, the development of three models (or in fact one model with three time horizons) was suggested.

The first is a long-term model that operates on a weekly basis and has a time horizon of one year. It will use long-term probabilistic forecasts of natural inflow and has the objective of maximizing the worth of power taking into account all constraints except flooding in Montreal, ($C_1 = 1$ and $C_2 = 0$). This will be used as a basis for measuring both the loss in power production and the reduction in peak flows at Carillon. In such a model, the state of the system is described by the amount of water in storage in the reservoirs and the decision variables are reservoir releases. In order to avoid unreaslistic decisions at the end of a time horizon, targets of the state variable are included in the objective function. In this case, the targets will be long-term mean states of the reservoirs derived from historic data.

The second is a mid-term model which has a time horizon of about four months in order to include the entire freshet. This also is a weekly model and uses a probabilistic forecast, but this time the composite objective function is used. The model can be run with various combinations of the coefficients C_1 and C_2 and the output should provide a comparison of losses in power production and reduction in flood damages in the Montreal Region. The intent is to provide all the data necessary to enable a decision on the operation of the river during the freshet. In this case, target states of the reservoirs would be those indicated by the long-term model at the end of the mid-term horizon.

The third model has a short-term time horizon of about two weeks and will operate on a daily basis. It will use real-time forecasts of natural inflow and have a fixed composite objective function depending on the decision made at the mid-term stage. This model will assist in daily operation of the reservoirs in the basin taking account of the overall objective. A schematic representation of this approach is shown in Figure 1.

Physical System

The Ottawa River basin is a very complex system containing a large number of reservoirs built and operated for various purposes. In order to model such a system, it is necessary to ignore many of the smaller storages which have little effect on flood peaks or power production. Figure 2 shows a schematic representation of the main rivers, reservoirs and generating stations in the basin. This representation was simplified by eliminating many of the small storages and generating stations and combining the effects of others. Figure 3 is an example of the system to be modelled. It should be noted that this configuration is preliminary and some of the details may be changed.

Data

To successfully model a complex system such as the Ottawa River basin, a considerable amount of data must be collected, analyzed and converted to a compatible format. Although much of the data had been gathered for previous modelling studies, it was decided not to use it for various reasons. There was considerable doubt as to the accuracy in some cases and the new approach used in this project required data in different forms. The only part that was not repeated was the collection of historical flows and levels. The existing data file, which contained this information for the period January 1956 to September 1974, was updated by adding two more years. This type of data will be used for long-term targets and testing of the model.

For each reservoir or generating station included in the model, data has been collected for the following and adapted to the required format:

Stage-storage relationships
Stage-maximum discharge relationships
Power-discharge relationships
Power-storage-discharge relationships

The first two are relatively straightforward but the power relationships are very complex and considerable thought has gone into the means of representing them in the model. This will be explained in more detail later in the report.

The following data are also required for modelling the system:

- routing parameters
- daily peak - weekly mean at Carillon
- absolute maximum and minimum levels of reservoirs
- operating maximum and minimum levels of reservoirs
- local constraints on flooding, recreation, navigation, maximum and minimum discharges
- seasonal values of power
- local power demand
- generating station maintenance schedules
- stage-damage curves for the Montreal Region, Maniwaki and Hull-Gatineau.

Also to be considered are two cases of special operation for Baskatong - Maniwaki and the Carillon headpond.

Development Strategy

Due to the size of the system to be modelled, the optimization required and the non-linear relationships involved, it was decided that a linear programming package with separable programming capability had to be used. This realization was reinforced by a review of the state-of-the-art in operational systems including a visit to several agencies in the U.S. currently working on the same type of project. It appears that the closest system to the Ottawa River now under development is that of the Tennessee Valley Authority which is mid-way through a 10 year program.

Because of the number of agencies involved in this study who may wish to use the final model for their own purpose, an early decision was made that it be as general as possible. This generality will probably be somewhat constrainted due to the time limitations on the project as will some "optional extras" that have been envisaged. It is thought, however, that there will be a section in the report pointing out possible improvements and/or refinements.

It was also decided to develop the model on IBM equipment, due to its general availability, but to keep in mind the possibility of transposing the final product to other computers. This led to the choice of the IBM commercial software package MPSX which represents the current state-of-the-art in linear and separable programming.

The Model

The operation of the real system is governed by the following relationships:

- water balance at each plant
- water balance between plants
- maximum discharge from a reservoir as a function of forebay elevation
- maximum flow through a set of turbines as a function of forebay elevation
- power production as a function of turbine characteristics, flow through turbines, forebay elevation, tailwater elevation and spill
- exceptions to above
  - maximum release or power production as a function of the state of downstream plants
- operating considerations
  - flood constraints at certain locations
  - various elevation-discharge relationships other than above
- overall objectives relating to energy production, flood damage, recreation, navigation, etc.

These relationships range from simple, exact identities to empirical tables of observed and/or estimated variable values.

Basically the modelling project consists of three phases

1) A detailed study of the physical characteristics and operation constraints of the Ottawa River system together with an evaluation of the available data.

2) Design of a linear/separable programming (LP/SP) model that can adequately represent the relationships and objectives and produce solutions that represent realistic operational guidelines.

3) Design and development of a modelling system that will:

  - Automatically convert the basic data into a form suitable for processing by MPSX,

  - Automatically guide MPSX toward cost effective solution paths, and

  - Automatically interpret the output from MPSX in such a way as to

    - automatically re-solve the basic problem in order to adjust for non-linearities

    - produce a user-oriented report which presents the recommended operating guidelines in as much detail as required.

In phase 1, the physical system was examined in depth and the data required was collected. In phase 2, a tentative outline of the MPSX model structure was developed including the definition of

- structural model variables by which the state of the system can be fully described, and

- constraints, limits and bounds on those variables which represent operational constraints in the real system.

There are, of course, many exceptional circumstances which arise due to either special operating problems at a specific plant or random occurrences of certain circumstances over time. It is therefore essential to provide a flexible framework in the modelling system to so that special variables and constraints can easily be specified and added to the basic model as required.

Phase 3, which is currently underway, involves the detailed design, coding and testing of the numerous programs and subroutines within the overall model structure.

Model Structure

The fundamental state variable in the system is the amount of water in storage at each of the reservoirs. Given a forecast of natural inflows and a knowledge of the storages, control over the system can be maintained by manipulating controllable releases from reservoirs. Plant discharges, therefore, are the decision variables on both the real system and the model. In some sense, the state of certain parts of the system - for example, stage in a channel - is a function of discharge. In this case, discharge can also be considered as a state variable; this, however, has absolutely no effect on the modelling approach.

Thus in general, the structural variables of the model are storages (volume of water in each reservoir at the end of each time period) and discharges (average discharge from each plant over each time period). The equations and bounds of the model are used to impose the appropriate constraints on these variables (water balance, physical relationships, etc.). There are a standard set of variables and rows defining the basic model and, as required, additional variables rows, bounds, etc. are generated automatically and added to the model prior to solution.

The proposed concept of the total system includes a model generator, the purpose of which is to mechanically generate a consistent, compact skeleton model of the river system. This requires as input only a set of simple descriptions of the individual stations. In order to provide a greater degree of generality, an interface is also provided to allow the generation of special relationships as exceptions to the normal case (e.g. forebay - tailwater elevation interaction between adjacent plants).

Inputs

The input data required for the skeleton generator include:

- descriptive information
    - titles
    - number of time periods
    - length of each time period
- configuration information
    - plant types
    - number of release channels
    - channel types (free, controlled, turbine)
    - plant names
- linkages
    - upstream and downstream plants
    - routing parameters
    - fixed delays
- stage-discharge relationships
    - coordinates of linear segments approximating the curves

- stage-storage relationships
  - physical upper and lower limits
  - coefficients of polynomial equations

- power-storage-discharge relationships
  - tables of coordinates of linear segments of power-discharge relationships for each fixed value of storage
  - downstream storage affecting tailwater
  - storage patterns

Using these fixed data, the skeleton model generator then performs two major tasks

- generates an MPSX LP/SP model with all rows, columns, bound vectors and fixed coefficients

- generates a communications file with all the master information for later use by other parts of the system.

Unless there are structural or characteristic changes to the system being modelled, this skeleton model generator need not be run again.

Each time the model is executed, the expansion/execution module must be run. This involves the following steps which are automatically performed as one computer job:

- The expansion module uses information from the communications file, inflow forecast file, current condition file and special constraint file to compute those LP/SP model coefficients which do not necessarily remain the same from one run to the next;

- These update coefficients are then merged with the skeleton model.

As was stated above, each time the model is run, there is a significant amount of input to be added to the skeleton model. This "run-time" input includes the following:

- starting date
- current states of the reservoirs throughout the system
- forecasted inflows for the time horizon to be modelled
- special constraints
- turbine maintenance schedules
- relative worth of power for each time period
- target states of the reservoirs
- objective function coefficients.

Outputs

Once the skeleton model and the run-time information are combined in the model expansion module, a current MPSX model is generated. This is then solved within MPSX and a tentative solution is produced. The next step is an evaluation phase in which the solution is checked for correctness of assumptions, feasibility (i.e., to see that no constraints have been violated) and optimality. If the solution is acceptable, it is analyzed and converted to a user-oriented report within the report generation module. The report can include any amount of information on the system such as the following:

- storage in each reservoir at the end of each time period
- average discharge in each channel over each time period
- power produced by each generating station during each time period
- total power production for each utility over the time horizon
- where appropriate, total worth of power production by each utility
- where appropriate, estimated flood damages
- for the short-term model, forecasted levels and discharges at specified locations.

In fact, the model is being designed to produce a report file that contains all the information available. Two or three standard output formats will be produced but, should a special format be required, it will be a simple matter to access the file.

Power Relationships

As the power relationships to be modelled are the most complicated, it is worthwhile to investigate in some detail the manner in which they will be considered.

The simplest are run-of-the-river plants were headpond storage is not really a factor. In most cases the forebay is kept at or close to maximum elevation in order to give as great a head as possible. In the model, the head will be fixed for these plants at a value corresponding to the long-term mean based on historical data. This, therefore, means that there is only one power-discharge curve to be represented in the system.

In past modelling studies of this type, some very simplified assumptions have been used to represent the power-discharge relationships in a linear form. In this case, however, the Working Group spent considerable time on this subject in an effort to develop an accurate, realistic relationship. As a result of this, a new technique was developed to enable the power-discharge relationships to be represented by linear segments which give results that should be more accurate than previous assumptions.

The key to the technique is the assumption that turbines are generally run at points of maximum efficiency. Thus, in Figure 5, which shows the relationship for a group of three turbines, the discharge would be at 0, Q_1, Q_2 or Q_3 in normal circumstances.

If the average discharge over a time period were, for example Q_i, which was half way between Q_2 and Q_3 ($Q_i = \frac{Q_2 + Q_3}{2}$), this would in fact be achieved by running at Q_2 for half the time and at Q_3 for the remainder. Thus a linear segment between (P_2, Q_2) and (P_3, Q_3) is truly representative of the average power, P_i, produced over that time period.

When the flow is greater than the maximum efficiency of all units ($Q > Q_3$), power increases up to P_{max} at Q_{Tmax}. After that, spill occurs and the power decreases. An assumption of linearity in this range should be sufficiently accurate for modelling purposes.

Now, the maximum efficiency point of the generating station at Q_3 is generally about 80% of maximum capacity of Q_{Tmax}. It is also assumed that the turbines may be run at the 90% value, thus there are two linear segments between Q_3 and Q_{Tmax}. In this case, therefore, a fairly accurate representation of the power-discharge relationship can be modelled with six linear segments.

For the case with a generating station at a reservoir, exactly the same representation is used for each of a range of storages in the reservoir (Figure 6). These data will be stored as tables in the model (P, Q, S) with a sufficient number of storage planes to enable a realistic linear interpolation between each.

In this case, however, an iteration process is required in the model to ensure that final power calculations are correct. For each time period, an average storage at a plant is assumed (S_i). The model is run, decisions (releases) made, and the balance equations are satisfied. The storage values at the end of the period are then known and the average computed (S_e). If the two averages are significantly different, then the computed power coefficients used by the model to select that solution may not be representative and a better solution may exist. Then the model is rerun with a new value of assumed average storage (may be $\frac{S_i + S_e}{2}$). This process is repeated until convergence is obtained within an acceptable tolerance.

A further new development to be used in this model is the provision of planned maintenance schedules when some units will be out of operation. Power-discharge-storage relationships will be provided for cases with one and two units out of service. Each will be identified by a pattern number and planned outages can be entered at run time by specifying the pattern number for each plant for each time period.

Aggregation

For each plant in the system to be modelled, there are a great number of variables to be stored and manipulated, making for a very large (and expensive) model. In order to reduce the size of the model wherever possible without significantly affecting the accuracy thereof, the possibility of aggregating or grouping reservoirs or generating stations was investigated. It has been found that this can be done with no detrimental effects in some cases, particularly for run-of-the-river generating stations with no significant intermediate inflows or delay. The same discharge passes through each plant so a composite power-discharge relationship can be developed that is as accurate as the sum of the individual ones. For reporting purposes, the power produced by each plant can be computed as a ratio of the total.

Aggregation can also be applied to simple reservoirs either in series or in parallel. Here, however, the simplifying assumption must be made that the reservoirs are operated in sequence (i.e. each contains the same percentage of total storage at the same time). This assumption may be too simplistic and in fact reduce the flexibility of the model. Specific cases are currently being investigated to determine whether or not such aggregation is feasible or desirable.

More complicated aggregation is also possible (for example where there is an intermediate natural inflow between plants) but it was decided not to pursue these at this time.

A further saving can be made by time aggregation, particularly in the long-term model. The normal time period used in this case is one week but, as great accuracy may not be required for the last few months of a run, this can be increased to two, three or four weeks to reduce the overall model size.

Inflow Forecasts

Each version of the model will require forecasts of natural inflows at numerous points throughout the basin. Such forecasts must be available for the entire time horizon of the model being run, therefore they may be required for periods of up to one year.

A watershed model developed by Hydro Quebec (and expanded as part of the work of Subcommittee 2) apparently has the capability to provide long term forecasts for many levels of probability. For the short-term model, of course, the real-time deterministic forecast will be the most important and the probability aspect is not that relevant.

In the original approach, it was proposed that a series of forecasts be provided at three levels of probability - high, mean and low. Each series would then be run through the model and the mathematical expectancy computed. It is almost certain that the results of this process would be different from that obtained by using only a mean forecast because of the non-linearity of the system.

At present, this approach is undergoing considerable review by Subcommittee 1 and a decision on the matter has not been reached. It may be necessary to await the completion of the model and undertake some sensitivity analyses to assist in a final decision.

Studies

The first real use of the modelling system upon completion will be in studying the effects on flooding and power production of the various possibilities of additional storage that have been identified in the Ottawa River basin. In order to undertake this task, data will have to be prepared on stage-storage, stage-discharge relationships, etc. for each of the proposed sites. It will then be a simple matter to fit them into the system by adding them to the skeleton generator. Series of data can then be selected to run through the model with and without the new storages for comparison.

Operation

In the original approach, the proposed real-time operation of the modelling system would be as follows:

- The long-term model is run with the objective of optimizing the system for power production ($C_1 = 1$, $C_2 = 0$) whilst satisfying all the local constraints.

- The mid-term model is run using targets established by the long-term run. A parametric programming capability provided by MPSX is used to automatically vary the coefficients of the objective function throughout a given range. For example, runs are made for:

$C_1 = .9, \quad C_2 = .1$

$C_1 = .8, \quad C_2 = .2$

$\vdots$

$C_1 = 0 \quad C_2 = 1$

For each of these runs, the worth of power production and estimated flood damages are computed and presented in tabular form. At this point, a decision is made on which of the options to choose based on flood damages, power losses and, of course, political considerations.

- The short-term model is then run with a fixed composite objective function based on that decision to guide the daily operation. This model would also serve the purpose of providing the forecast flows and levels at relevant locations throughout the basin.
- Several days or a week later, the entire process is repeated, a new decision made (based on updated forecasts) and new operation procedures suggested.

It now appears, however, that this procedure is not operationally viable. It has been pointed out that it is not realistic to change operating policy in the middle of a freshet; in fact it should be established long before the freshet starts.

The current thinking, therefore, is that the model be run on a series of years of historical data using the variable composite objective function. Analyses of the results would then be carried out to determine the effects of varying the coefficients on power production and flood damages. A decision would be made and the policy adhered to for the following year regardless of the inflows.

This has the advantage of eliminating the great uncertainty associated with forecasts, but appears to have the disadvantage of not using current information (during the freshet) to the best advantage. This is also currently under review by the Subcommittee and a decision is expected shortly.

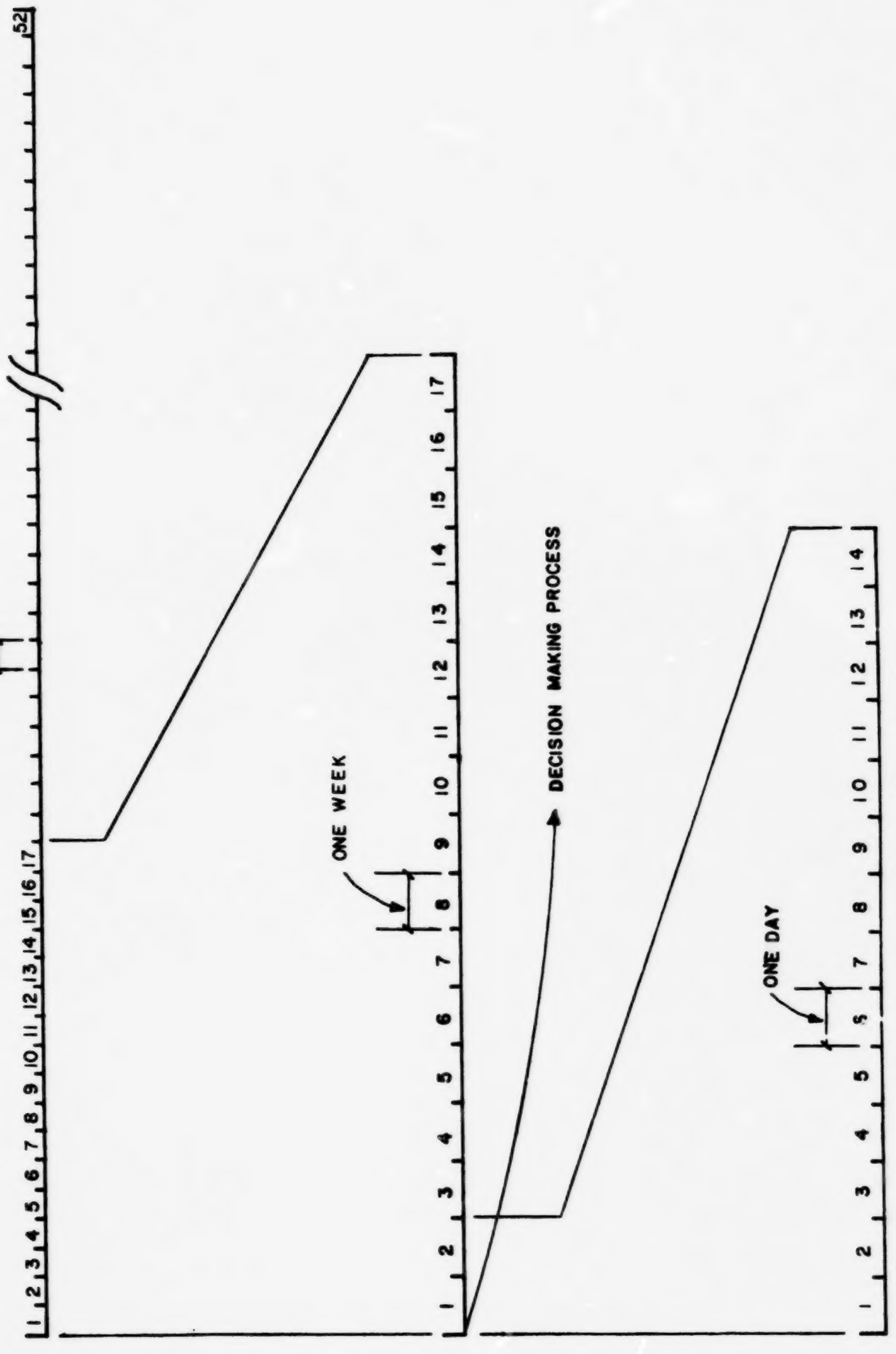
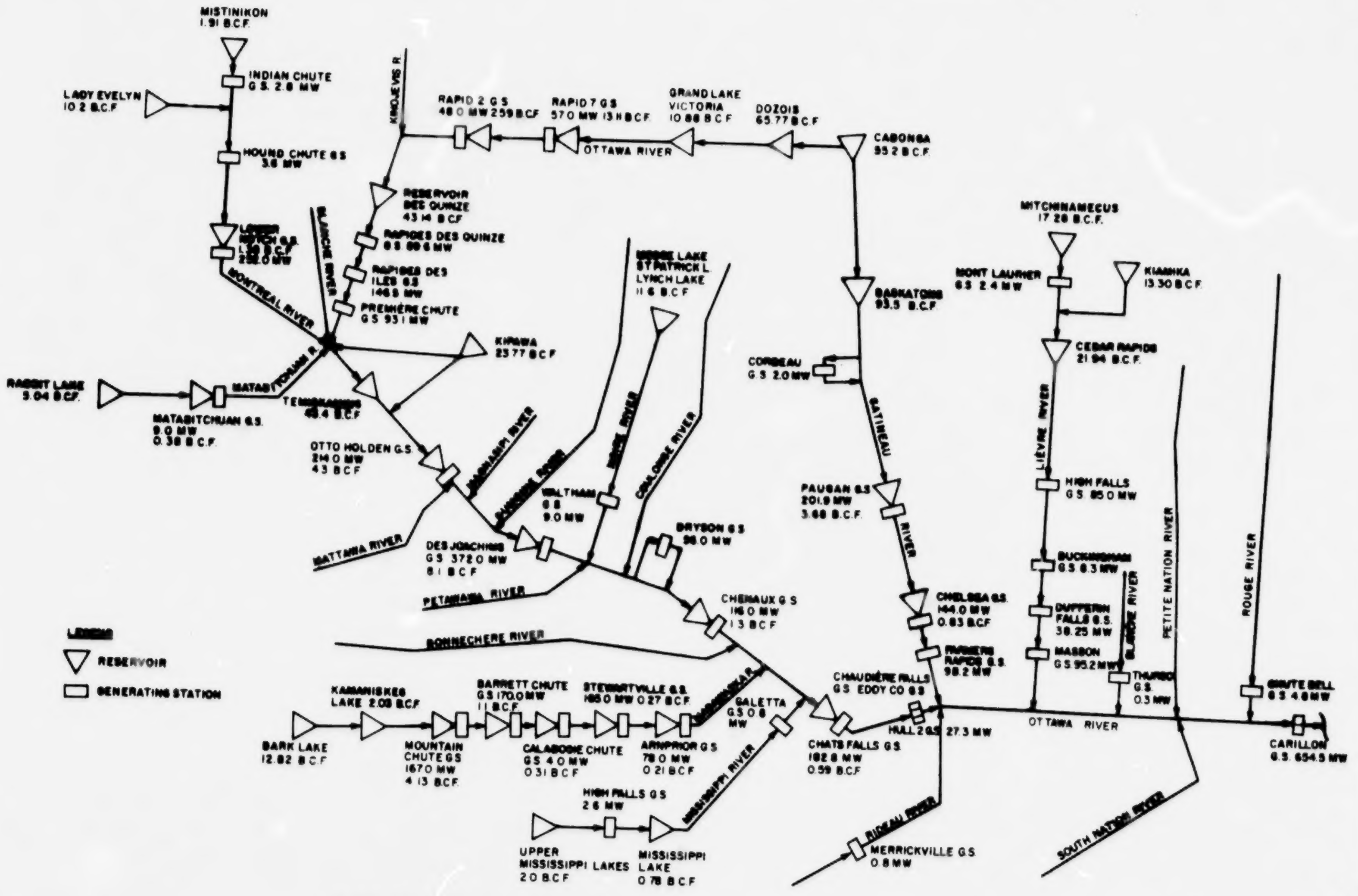

ONE WEEK
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17
52
ONE WEEK
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17
DECISION MAKING PROCESS
ONE DAY
1 2 3 4 5 6 7 8 9 10 11 12 13 14

FIGURE 1

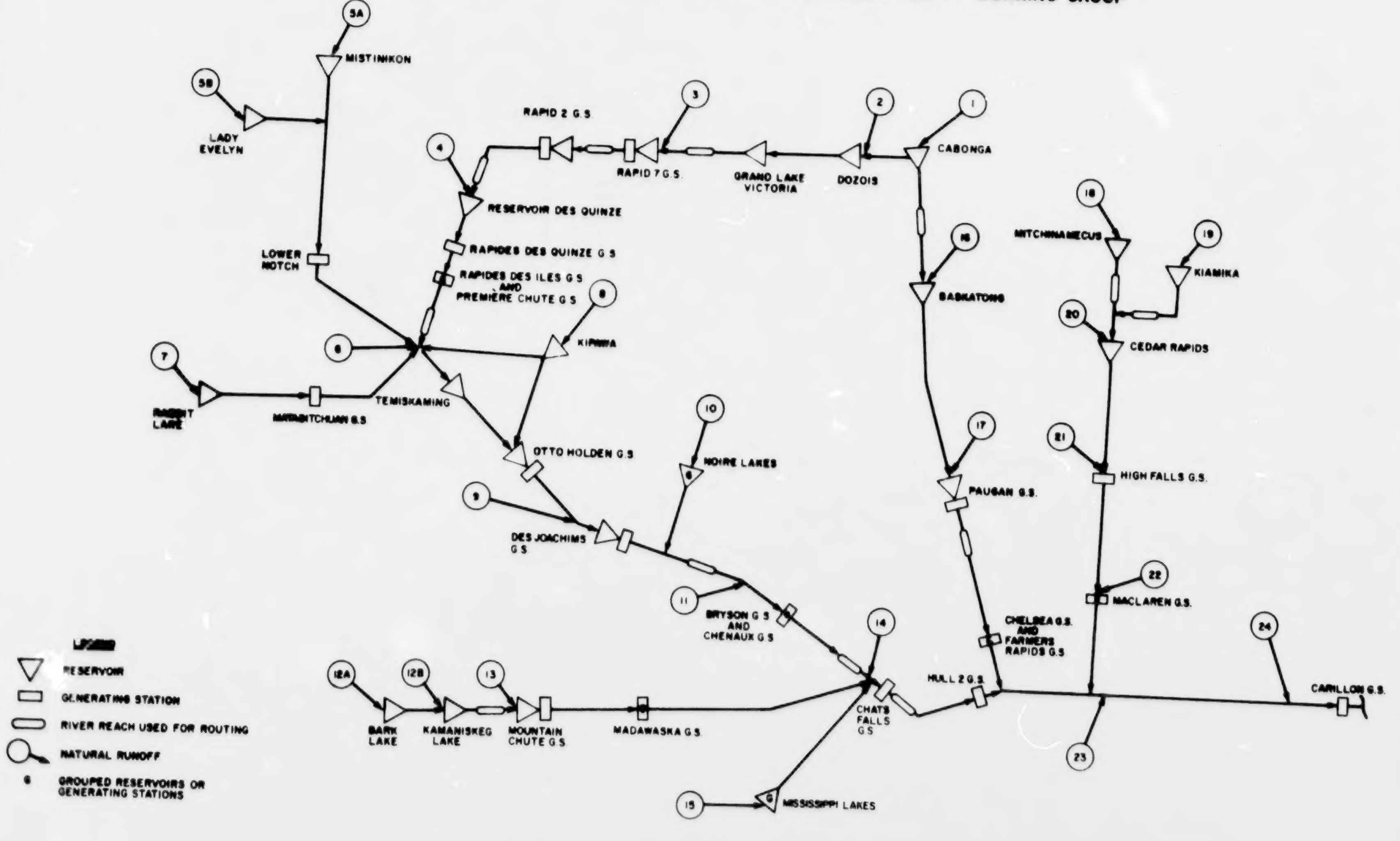
OTTAWA RIVER REGULATION PLANNING COMMITTEE SUBCOMMITTEE I - WORKING GROUP


MISTINIKON 1.91 B.C.F.
INDIAN CHUTE G.S. 2.8 MW
LADY EVELYN 10.2 B.C.F.
HOUND CHUTE G.S. 3.6 MW
LOWER NOTCH G.S. 1.30 B.C.F. 252.0 MW
MONTREAL RIVER
BLANCHE RIVER
KINOJEVIS R.
RAPID 2 G.S. 48.0 MW 259 B.C.F.
RAPID 7 G.S. 57.0 MW 13 H B.C.F.
GRAND LAKE VICTORIA 10.88 B.C.F.
DOZOIS 65.77 B.C.F.
CABONGA 55.2 B.C.F.
OTTAWA RIVER
RESERVOIR DES QUINZE 43.14 B.C.F.
RAPIDES DES QUINZE G.S. 88.6 MW
RAPIDES DES ILES G.S. 146.5 MW
PREMIÈRE CHUTE G.S. 93.1 MW
KIPAWA 23.77 B.C.F.
RABBIT LAKE 3.04 B.C.F.
MATABITCHUAN G.S. 9.0 MW 0.38 B.C.F.
MATABITCHUAN R.
TEMISKAMING 45.4 B.C.F.
OTTO HOLDEN G.S. 240 MW 4.3 B.C.F.
KIPAWA RIVER
MAGANASIPI RIVER
DUMOINE RIVER
NOIRE RIVER
COULONGE RIVER
PIERRE LAKE ST PATRICK L. LYNCH LAKE 11.6 B.C.F.
WALTHAM G.S. 9.0 MW
BRYSON G.S. 56.0 MW
MATTAWA RIVER
DES JOACHIMS G.S. 372.0 MW 8.1 B.C.F.
PETAWAWA RIVER
CHENAUX G.S. 116.0 MW 1.3 B.C.F.
BONNECHERE RIVER
LEGEND
RESERVOIR
GENERATING STATION
BARK LAKE 12.82 B.C.F.
KAMANISKEG LAKE 2.03 B.C.F.
MOUNTAIN CHUTE G.S. 167.0 MW 4.13 B.C.F.
BARRETT CHUTE G.S. 170.0 MW 1.1 B.C.F.
CALABOGIE CHUTE G.S. 4.0 MW 0.31 B.C.F.
STEWARTVILLE G.S. 165.0 MW 0.27 B.C.F.
ARNPRIOR G.S. 78.0 MW 0.21 B.C.F.
MADAWASKA R.
GALETTA G.S. 0.8 MW
HIGH FALLS G.S. 2.6 MW
UPPER MISSISSIPPI LAKES 2.0 B.C.F.
MISSISSIPPI LAKE 0.78 B.C.F.
MISSISSIPPI RIVER
BASKATONG 93.5 B.C.F.
CORBEAU G.S. 2.0 MW
GATINEAU RIVER
PAUGAN G.S. 201.9 MW 3.68 B.C.F.
CHELSEA G.S. 144.0 MW 0.83 B.C.F.
FARMERS RAPIDS G.S. 98.2 MW
CHAUDIÈRE FALLS G.S. EDDY CO G.S.
CHATS FALLS G.S. 182.8 MW 0.59 B.C.F.
HULL 2 G.S. 27.3 MW
RIDEAU RIVER
MERRICKVILLE G.S. 0.8 MW
MITCHINAMECUS 17.28 B.C.F.
MONT LAURIER G.S. 2.4 MW
KIAMIKA 13.30 B.C.F.
CEDAR RAPIDS 21.94 B.C.F.
LIÈVRE RIVER
HIGH FALLS G.S. 85.0 MW
BUCKINGHAM G.S. 8.3 MW
DUFFERIN FALLS G.S. 38.25 MW
MASSON G.S. 95.2 MW
BLANCHE RIVER
THURSO G.S. 0.3 MW
PETITE NATION RIVER
ROUGE RIVER
CHUTE BELL G.S. 4.8 MW
OTTAWA RIVER
SOUTH NATION RIVER
CARILLON G.S. 654.5 MW

REPRESENTATION OF THE OTTAWA RIVER HYDRAULIC SYSTEM

FIGURE 2

OTTAWA RIVER REGULATION PLANNING COMMITTEE SUBCOMMITTEE I - WORKING GROUP



5A
MISTINIKON
5B
LADY EVELYN
LOWER NOTCH
RAPID 2 G.S.
3
RAPID 7 G.S.
GRAND LAKE VICTORIA
2
DOZOIS
1
CABONGA
4
RESERVOIR DES QUINZE
RAPIDES DES QUINZE G.S.
RAPIDES DES ILES G.S. AND PREMIÈRE CHUTE G.S.
8
KIPAWA
6
7
RABBIT LAKE
MATABITCHUAN G.S.
TEMISKAMING
OTTO HOLDEN G.S.
10
NOIRE LAKES
9
DES JOACHIMS G.S.
11
BRYSON G.S. AND CHENAUX G.S.
14
CHATS FALLS G.S.
16
BASKATONG
17
PAUGAN G.S.
CHELSEA G.S. AND FARMERS RAPIDS G.S.
HULL 2 G.S.
18
MITCHINAMECUS
19
KIAMIKA
20
CEDAR RAPIDS
21
HIGH FALLS G.S.
22
MACLAREN G.S.
23
24
CARILLON G.S.
12A
BARK LAKE
12B
KAMANISKEG LAKE
13
MOUNTAIN CHUTE G.S.
MADAWASKA G.S.
15
MISSISSIPPI LAKES
LEGEND
RESERVOIR
GENERATING STATION
RIVER REACH USED FOR ROUTING
NATURAL RUNOFF
GROUPED RESERVOIRS OR GENERATING STATIONS

REPRESENTATION OF THE OTTAWA RIVER SHORT TERM MODEL

FIGURE 3

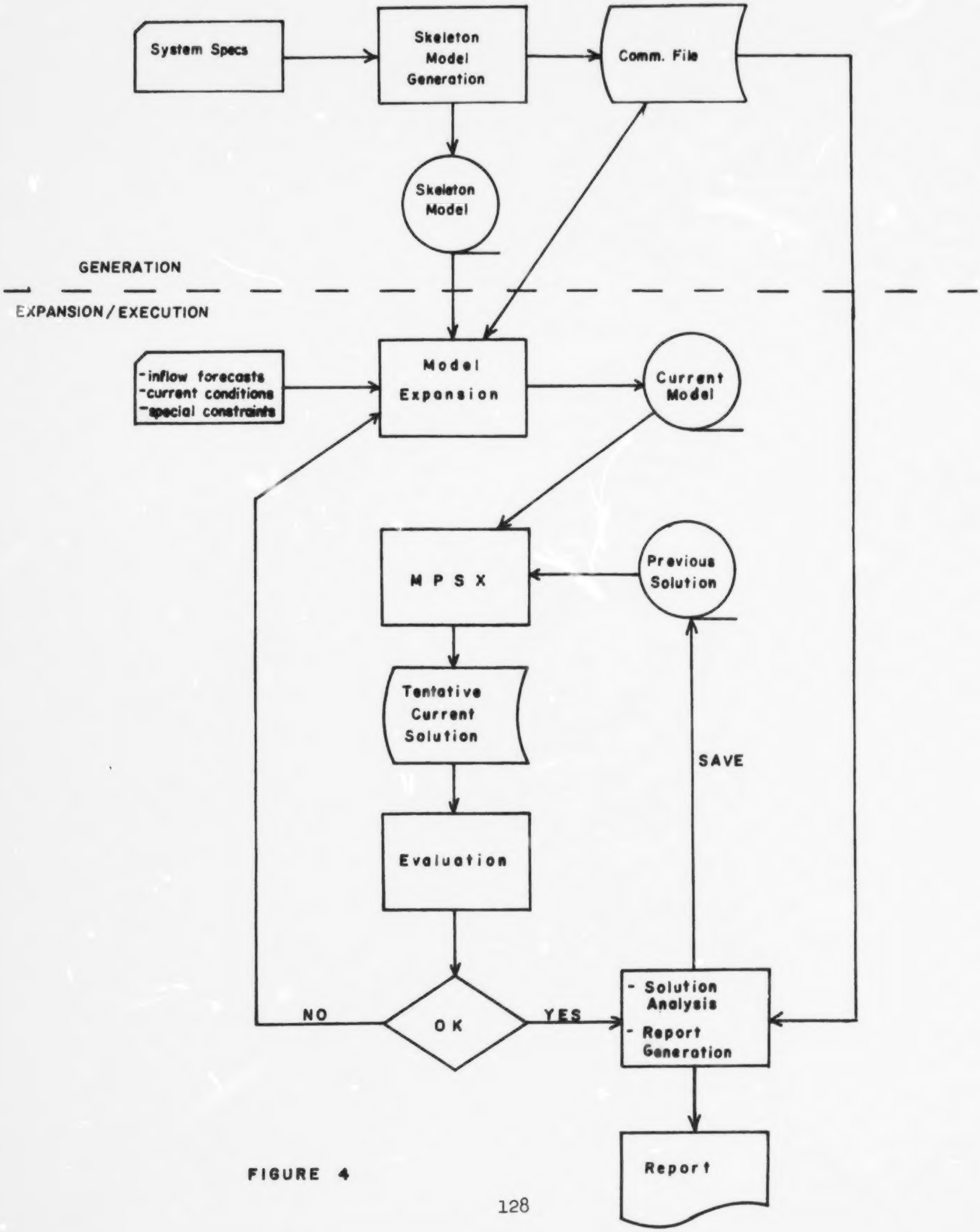

System Specs
Skeleton Model Generation
Comm. File
Skeleton Model
GENERATION
EXPANSION / EXECUTION
-inflow forecasts
-current conditions
-special constraints
Model Expansion
Current Model
MPSX
Previous Solution
Tentative Current Solution
SAVE
Evaluation
NO
OK
YES
- Solution Analysis
- Report Generation
Report

FIGURE 4

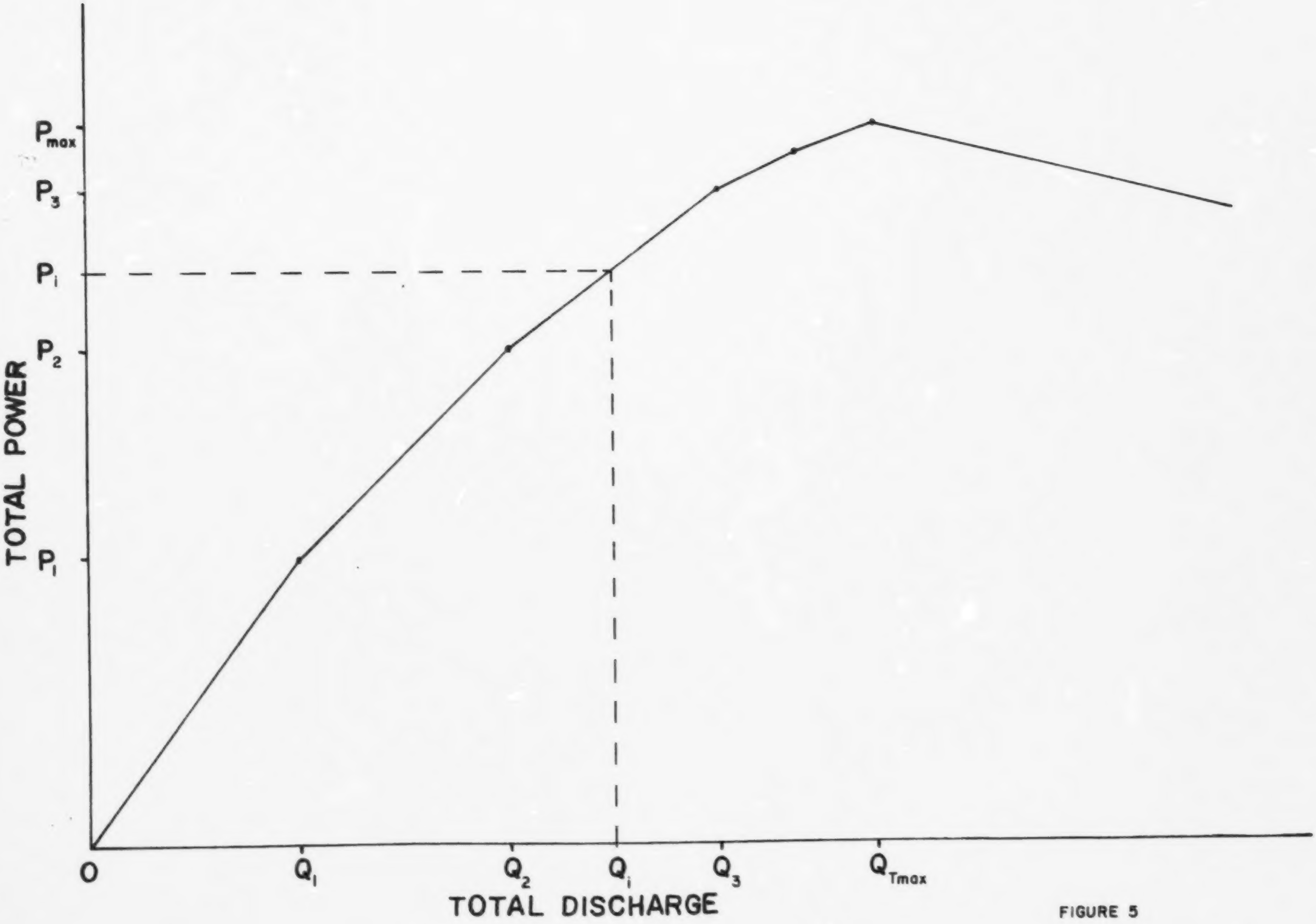

TOTAL POWER
P_{max}
P_3
P_i
P_2
P_1
O
Q_1
Q_2
Q_i
Q_3
Q_{Tmax}
TOTAL DISCHARGE

FIGURE 5

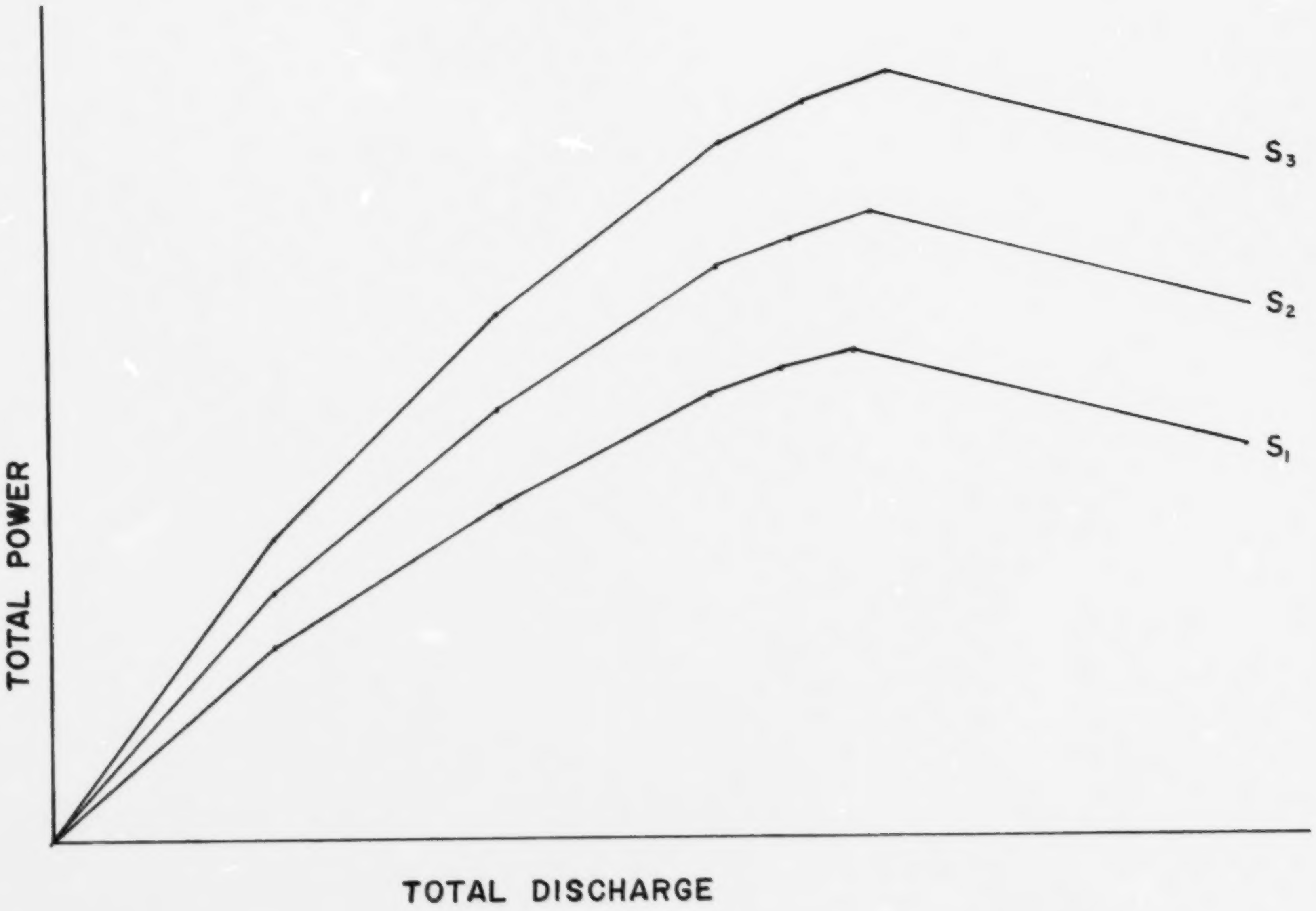


FIGURE 6

FLOOD DAMAGE REDUCTION PROGRAM
TECHNICAL WORKSHOP
OTTAWA MAY 23-25, 1979

1-2 Steady State Water Surface Profile Models HEC 2 Application at Moose Jaw - Input Requirements and Model Performance

- R.D. Hofer

INTRODUCTION

IWD, Western and Northern Region entered into an agreement with the Saskatchewan Department of Environment to conduct hydraulic investigations of the Moose Jaw River, Spring Creek and Thunder Creek under the Canada-Saskatchewan Flood Damage Reduction Agreement. The study area is shown on Figure 1. The contract agreement was originally for $36,100 but the agreement has subsequently been increased to $40,700 to allow for extensions of the work. A further extension is expected shortly to cover additional work on Spring Creek.

PREVIOUS MODELLING

In 1972 and 1973 the Western Regional Office of Water Planning and Management Branch carried out a $10,000 hydrologic investigation of the Moose Jaw River at the City of Moose Jaw for the Qu'Appelle Basin Study. The investigation developed flood hydrographs, and routed these hydrographs through proposed reservoirs upstream of the City of Moose Jaw. Estimated peak flows under regulated and unregulated conditions were used to estimate water surface profiles through the City. A water supply study was also carried out for three reservoir alternatives.

The Moose Jaw River cross sections changed constantly along the reach. In addition there were several bridges, road fills and dams across the river. The river was analyzed at 56 cross sections using a backwater program entitled HY-13 that was obtained from the Prairie Farm Rehabilitation Administration. This program was very useful and could apply different roughness values to the channel and overbank sections. Coding however, required too much detail, and pressure flow conditions at bridges and dams could not be modelled.

Several backwater runs were carried out for estimated peak flows for the 1:10, 1:25, 1:50, 1:100, 1:500 and the 1969, 1946 and probable maximum floods. Various structural alternatives were considered including constriction removal, channel improvement and dyking. There was a continued involvement with the many Qu'Appelle Implementation Program alternatives for the lowering of water levels along the Moose Jaw River.

THE HEC-2 INVESTIGATIONS

The Flood Damage Reduction Program contract to conduct hydraulic investigations of the Moose Jaw River, Thunder Creek, and Spring Creek was signed on March 8, 1978. This was the first hydraulic contract to be carried out under the Canada-Saskatchewan Flood Damage Reduction Agreement and was considered to be the pilot project for the province. The initial contract called for the development of 1:100 and 1:500 floodlines along the Moose Jaw River, Thunder Creek and Spring Creek. A floodway along the Moose Jaw River was also called for. The HEC-2 model was acquired for this project in order to carry out the stream flow modelling.

In order to develop the floodway along the Moose Jaw River three criteria were used; the one metre depth, one metre per second velocity and the 30 cm floodway criteria were applied to the channel as modified by the structural measures proposed under the Qu'Appelle implementation Program. Floodlines were drawn up on paper prints and forwarded to the Technical and Steering Committees for review. These Committees reviewed the lines on the basis of provincial requirements and also discussed them with City of Moose Jaw officials for local considerations.

The Qu'Appelle Implementation Program structural measures gave rise to difficulties. There were two main projects designed to reduce the water surface profiles through the city that could not be completed. The first project was the removal of the High Pressure Dam's piers and road deck. This project could not proceed because the City did not follow the required procedure for closure of the road connections. The second project was delayed because mercury contamination of the river bottom was discovered along the reach of the Moose Jaw River that was to be dredged. As a result of these delays, the channel configuration was changed to reflect only existing conditions. The final work plan for Moose Jaw provided the 1:500 flood levels under encroachment conditions in the existing channel as well as the 1974 flood which was the largest recorded.

BASIC DATA ACQUISITION

Over the duration of the hydraulic contract the City of Moose Jaw has been busy improving channel flow conditions so that some of the flood conditions experienced in 1974 would not recurr. This continual change forced the study group to keep up to date on construction activities in the river channels.

Since Moose Jaw has been the focus of considerable federal and provincial government survey activity much cross-sectional data was available. It became readily apparent however, that much of the survey work contained mathematical errors. Field books and cross-sectional drawings had to be corrected. Data coding for HEC-2 program was also reviewed. Most of the corrections had little effect on the water surface profiles. However, some resulted in significantly raised or lowered water surfaces.

Fortunately numerous air and ground photographs were taken of the Moose Jaw River during the peak flow period of the 1974 flood. These photographs were used extensively for calibration of the three streams. The photographs provided an excellent check of observed stage levels and provided a means of estimating levels in locations not covered by the water level observations. Photographs of structures were particularly useful in establishing head losses in locations where flows crossed road approaches adjacent to bridge, culvert or dam structures.

Without this information, the "n" values determined for river reaches would have been unrealistically high. In spite of the large number of available photographs, many were useless for calibration purposes because the photographs were not dated or otherwise identified.

Maximum flood crest observations taken by City and Branch staff were very useful in the calibration of river reaches. However, these values had to be appraised to determine if they represented the water surface under free flow conditions or a transient water surface caused by channel blockage or surges. Along Thunder Creek, the maximum flood levels resulted when debris from industrial activities upstream of Moose Jaw lodged in bridge structures and raised water levels. These crest values could not be used for calibration of open water conditions so water levels were determined from photographs taken the day after the flood peak when the bridge blockages had been cleared.

Water levels from the 1:500 flood calculated by the HEC-2 program were found to be lower than the observed 1974 levels in uptown Moose Jaw under the restricted flow conditions. The HEC-2 model was calibrated successfully for several types of structures which included overflow dam structures (with stoplogs), rock weirs, clear span bridges, bridges supported by piles, arch bridges, and culverts. Adjustments had to be made at certain locations when water levels reached the top of bridge decks and the water passed through critical depth while moving across road approaches.

FLOODWAY DELINEATION

The one metre depth line was determined for the entire length of the Moose Jaw River and Thunder Creek study areas. Positioning this line was straight-forward because the line was simply one metre below the 1:500 water surface line. The one metre velocity line was also provided by the HEC-2 program by station-velocity summaries and it was plotted directly at each cross section. At some cross sections the one metre velocity station was obtained by interpolation. The 30 cm rise routine was used by employing the HEC-2 Method 4 to establish the encroachment stations at the cross sections.

Paper prints were prepared for the three criteria. This information was reviewed by the Technical and Steering Committees who derived a single floodway line that they felt would best meet the hydraulic criteria while recognizing the practical requirements of the existing community. This floodline was remodelled using Method 1 for those cross-sections where the line was shifted from the original 30 cm encroachment line. The remodelled line was then drawn on the mylars. An example of the final floodway is shown in Figure 2.

MODELLING PROBLEMS

Most of the routines found in the HEC-2 program were used directly during the Moose Jaw Study. However, the "CI-Channel Modification Due to Excavation" routine did not model the system adequately. A routine entitled CHIMP2 was written by D.J. Richards and D. Prytula to add channel improvements to the natural cross-sections. The most important feature of this routine is that it positioned the centre of the construction zone so that the area excavated was minimized.

The determination of the floodway routine was made with Method 4. Two problems were encountered using this routine. The program occasionally produced encroachment stations that were considerably outside the flooded area. Why this happened remains a mystery.

The second problem was that the water surface would not rise at locations where there was a significant constriction of the flow area. This was not a problem caused by the model but rather by channel conditions. At some locations the flow became supercritical and differences between the encroachment water surface and the initial water surface were near zero or negative. For this reason the 30 cm rise criteria could not be used on the West and North Arms of Spring Creek.

CONCLUSIONS

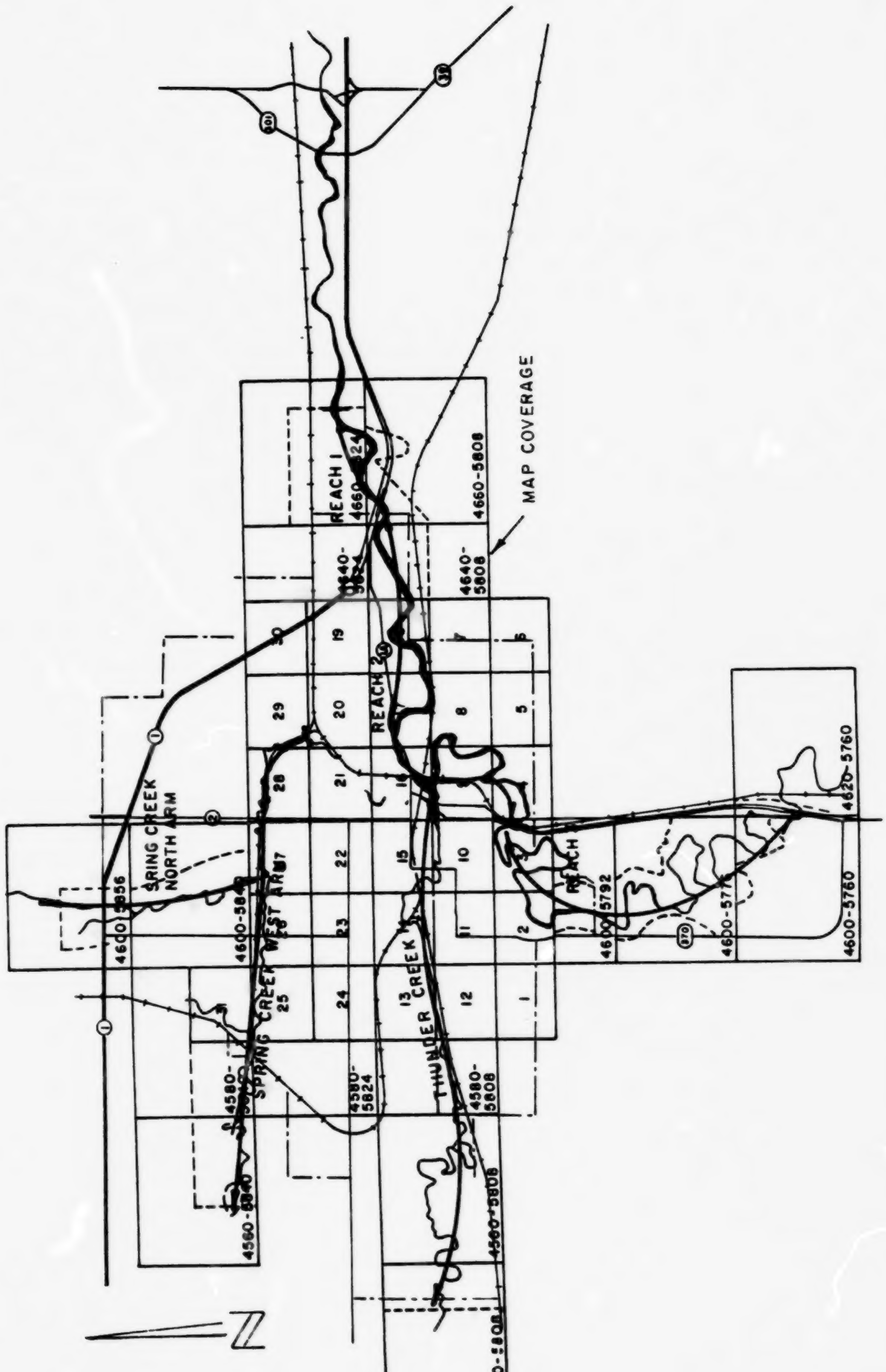
The HEC-2 model was very useful in determining water surfaces through Moose Jaw. It was fortunate that many water level observations were available since n values were often different than expected. With small cross-sections it was found that the channel improvement routine contained in the HEC-2 model did not minimize the area to be excavated. The CHIMP2 program was prepared to make up for this deficiency. The HEC-2 model's Method 4 encroachment routine on occasion yields encroachment stations beyond the flooded area. This problem should be rectified. In summary the HEC-2 model provides useful information but requires well controlled data to provide accurate results.

REFERENCES

U.S. Army Corps of Engineers HEC-2 Water Surface Profiles User Manual, November 1976.

Prytula, David, Water Planning and Management, CHIMP2 documentation, August 8, 1977.

CITY OF MOOSE JAW


SPRING CREEK NORTH ARM
SPRING CREEK WEST ARM
THUNDER CREEK
REACH 1
REACH 2
REACH
MAP COVERAGE
4600-5856
4600-5840
4580-5840
4560-5840
4580-5824
4580-5808
4560-5808
4540-5808
4640-5824
4660-5824
4640-5808
4660-5808
4600-5792
4600-5776
4600-5760
4620-5760
1
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N

FIGURE I

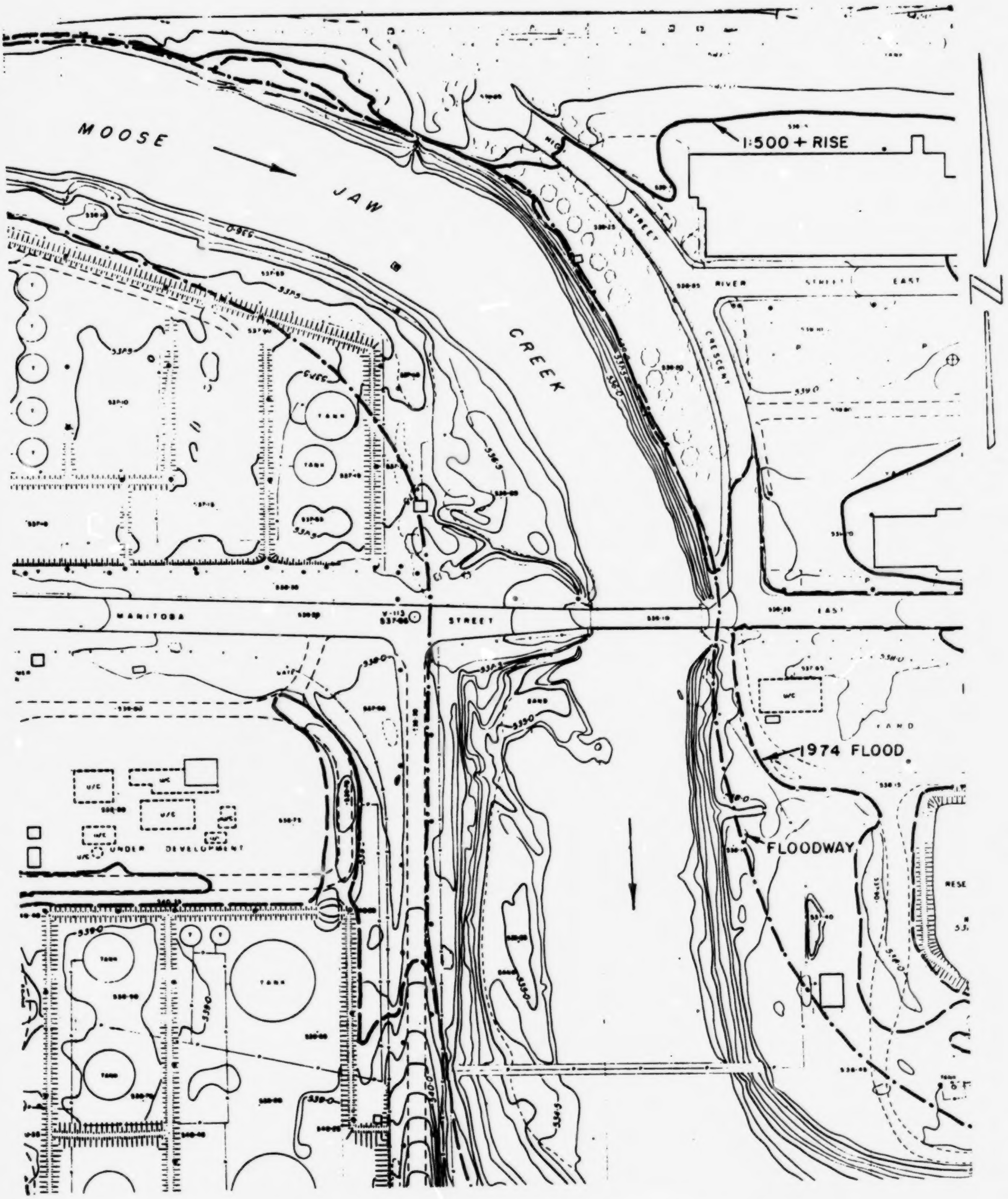
MOOSE
JAW
CREEK
1:500 + RISE
STREET
RIVER
STREET
EAST
CRESCENT
MANITOBA
STREET
EAST
TANK
UNDER DEVELOPMENT
1974 FLOOD
FLOODWAY
N

FIGURE 2

MATHEMATICAL MODELLING OF THE FRASER RIVER SYSTEM

ABSTRACT

A. THE NEED FOR HYDRODYNAMIC MODELLING IN THE LOWER FRASER RIVER

The Fraser River is the largest and most important waterway leading to the Pacific Coast of Canada. The river has a total length of approximately 850 miles. Although the river serves many functions, the most important is its use as a transportation corridor for industrial development. Navigation by deep sea vessels is confined to the seaward 22 miles of the main arm with shallow draft traffic utilizing the remainder of the Lower Fraser. The average freshet flow in the Fraser is 250,000 cfs with maximum daily discharges of 536,000 cfs.

A phenomenon associated with all natural bed rivers is sedimentation. Sediment, is a product of erosion of the earths crust. Fluvial sediment is eroded material transported by water towards the oceans. In the Fraser River, an average of 20 million tons of material enters the estuary annually. A substantial portion of this is deposited in the navigational channel constituting a hazard to shipping.

The Department of Public Works of Canada have been responsible for maintenance and improvements to the harbour since the late 1880's. In an attempt to keep the shipping channel free for navigation, there has been in excess of 120,000,000 cubic yards dredged and more than 25 miles of bank protection built.

In 1964, the Department of Public Works requested a sediment survey program on the Fraser River. Flow and sediment transport data were required for assessing problems related to the navigation channel.

A monitoring program was set up with a network of four (4) major sediment stations on the Fraser at Hope, Agassiz, Mission and Port Mann and five (5) supplementary stations on tributaries. One of the initial objectives was to produce a total volume of sediment figure entering the estuary. A necessary components for the calculation of total sediment load is the mean daily stream discharge.

At this point of the program a problem surfaced, continuous discharge information was not available at the two downstream stations on the Fraser River, namely Fraser River at Mission and Fraser River at Port Mann pumping station. The tidal effect on the river creates unsteady flow conditions in the Fraser for 60 miles upstream of the mouth. As a result, discharges cannot be measured at these two sites using conventional methods.

Discharges for the Fraser River at Mission are significantly influenced by backwater due to a tidal effect at low flows. Discharges for flows above the arbitrary 10 foot gauge height are only slightly affected. On the other hand, the Fraser River at Port Mann pumping station is under tidal influence at all stages with complete flow reversals at low water discharges. As a result, a non conventional method of computing discharges at these two stations is required.

The initial attempt at unconventional discharge computations was the Cubature Method. This method requires a steady base flow and approximate tidal effect using a double wedge system for storage. Although this method is being used at Mission for providing approximations of flow it is only a temporary measure. At the Port Mann pumping station flow reversals make it virtually impossible to use the cubature method.

Two hydrodynamic models were tested as alternate methods of providing discharge information. The first was the Power Series Method which assumed constant friction. It proved unsatisfactory for two reasons: it would not handle reverse flow and the friction factor is certainly not a constant. The second model was the Marine Sciences Fraser River Model. It also failed to provide discharge information to within specified limits. The average error for this model was ±15 percent.

The need for a hydrodynamic model which estimates water discharges at the Fraser River stations of Mission and Port Mann pumping station still exists. The estimate must be within Water Survey of Canada's standard of accuracy which is ±5% of the measured discharge.

There are many possible uses for an unsteady state model developed on the lower Fraser. First of all, data previously collected could be run and interpreted (Port Mann pumping station flows have not been published since 1972). Sediment data could be worked up for previous data. The model may be used in flood forecasting and sediment load prediction. If possible, the model may be extended down the estuary towards the mouth of the Fraser.

PHIL ZRYMIAK

B. ONE-DIMENSIONAL HYDRODYNAMIC MODEL OF THE LOWER FRASER RIVER

A one-dimensional finite-difference hydrodynamic model has been applied to the complex channel system of the Lower Fraser River. The basic model equations are based upon the St. Venant long wave equations for open channels. The resistance equations are described by the empirical "Manning n" translated into the equivalent Chezy coefficient.

The input data is a set of approximately 700 cross-sections collected by DPW in 1962 and 1976. The boundary points used by the model are hydrometric gauging stations at Mission, Port Mann and Pitt Lake. The total area covered is approximately 50 miles of river, of which is included a 15 mile stretch of the Pitt River.

Verification and calibration was carried out by initially dividing the river into two areas, each having boundary gauging stations and one interior check or calibration station. The data input problems mainly stemmed from changes in river cross-sections due to dredging that has occurred since 1962. Additional field surveys have been requested at specific check locations and these are to be included in final calibrations of the respective reaches of the river.

Initial data analysis has revealed that tidal effects extended up to Mission, the most upstream station, and thus reduced the possibility of using discharge as a basic input at this upstream point.

Initial success using the model has been realized by calibrating to a discharge of 350,900 cfs discharge on July 12, 1976 and by simulating the June 4, 1976 river condition when 227,500 cfs was recorded. When calculations were performed on a 15 minute basis, a maximum deviation of 0.2 feet was found at the upstream check station at Whonock with a corresponding deviation of +4% of the average daily flow computed at Mission. The lower study section responded with a 0.2 foot deviation at Port Hammond and 0.1 foot at Port Coquitlam check stations.

With such encouraging results, the studies are continuing and various periods are to be studied in order to determine the range of accuracy possible in using this model.

MAURICE SYDOR

APPLICATION OF THE DWOPER MODEL TO THE LOWER SAINT JOHN RIVER

by

Harold S. Belore[1], Amrita K. Bhattacharyya[2], Craig T. Bishop[3]

ABSTRACT

An unsteady flood routing model, referred to as DWOPER (an acronym for Dynamic Wave Operational Model), has been applied along the lower Saint John River. The model, which is representative of the state-of-the-art in unsteady flow modelling was selected for this application after considering a number of available hydraulic routing techniques. The model accounts for factors such as the downstream effects of the Reversing Falls and flow over the Trans Canada Highway to large storage areas in the Grand Lake area. Model calibration and verification simulations resulted in a good reproduction of historical flood levels. Subsequently, flood profiles associated with the 1:20 and 1:100 year recurrence interval flood events were determined along the lower Saint John River from McKinley Ferry to Lower Jemseg.

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2. Water Resources Engineer, M. M. Dillon Limited, Toronto, formerly with James F. MacLaren Limited
3. Water Resources Engineer, James F. MacLaren Limited Willowdale, Ontario

APPLICATION OF THE DWOPER MODEL TO THE LOWER SAINT JOHN RIVER

INTRODUCTION

As part of the Canada-New Brunswick Flood Damage Reduction Program, a hydrotechnical study of the lower Saint John River has been carried out by MacLaren Atlantic Limited for the New Brunswick Department of the Environment. The main study objective was to select and apply an appropriate hydraulic routing model in order to determine the flood profiles with recurrence intervals of 1:20 and 1:100 years using the hydrologic findings of earlier investigations(1). The emphasis of the present study was concerned with calculating flood profiles for the river reach from McKinley Ferry to Lower Jemseg, a distance of about 80 kilometres. The reach from lower Jemseg to Saint John was also modelled in order to account for the influence of hydraulic factors, such as the tidal fluctuations at Saint John on the flood profiles in the study reach. The study area is depicted in Figure 1. An unsteady flood routing model, DWOPER (an acronym for Dynamic Wave Operational Model), was calibrated and utilized to compute the flood profiles(4).

The study consisted of three main parts, the first phase of which involved a review of pertinent background information and previous studies (2,7,8,9,10,11,13,14,16). Using this information, the review and selection of appropriate hydraulic routing models was also undertaken. The review of background information and preliminary model simulations (discussed below) lead to the identification of data voids and the associated implementation of a field program to collect relevant hydrologic, hydraulic and physical data.

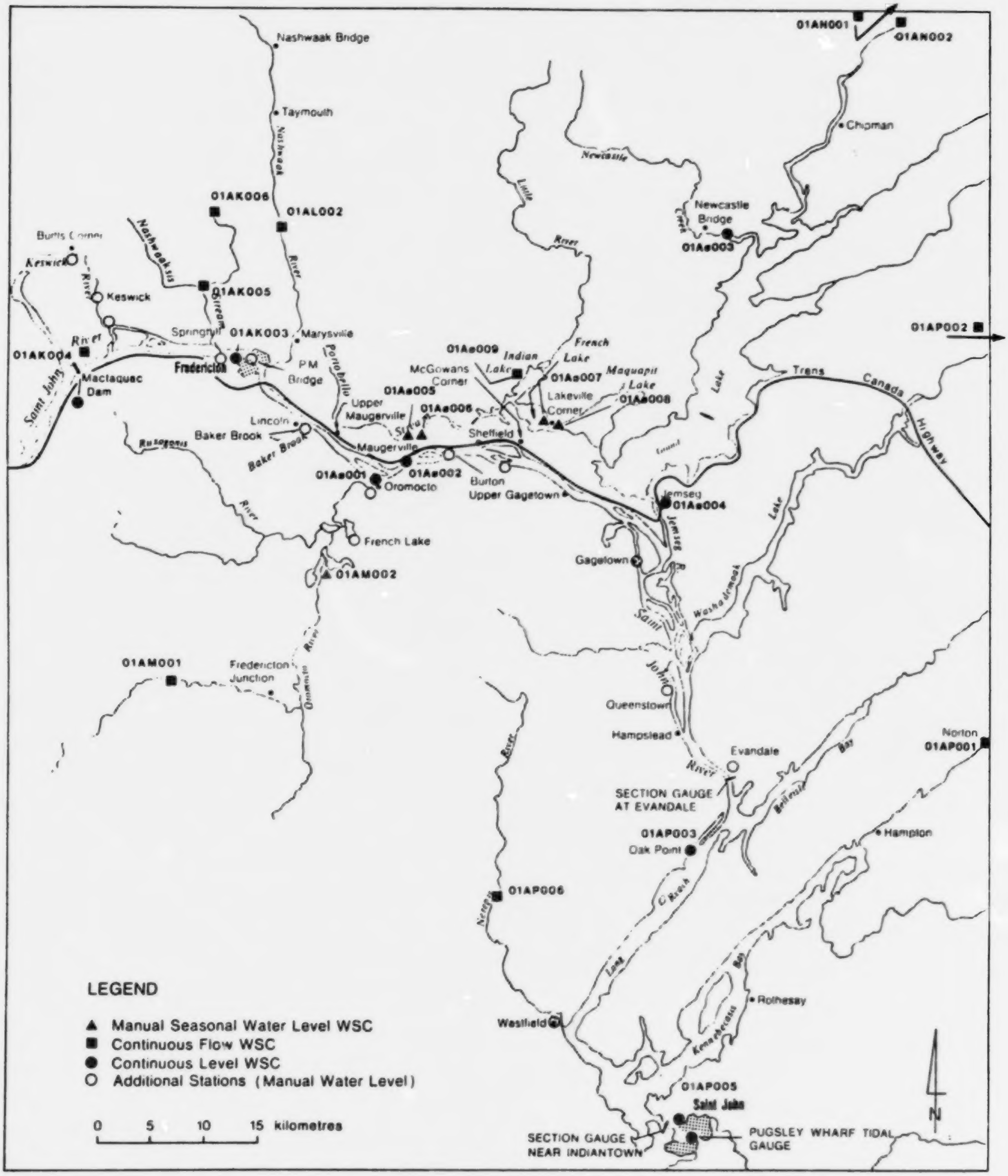
The second phase of the investigation involved the detailed compilation of existing data, and the collection of new data as necessary (12).

The third phase of the study involved the analysis and interpretation of the data, including calibration and verification of the selected mathematical model. This subsequently lead to the derivation of the 1:20 and 1:100 year flood profiles along the study reach.

This paper pertains mainly to the hydraulic modelling aspects of the study, with reference to the data collection and compilation aspects where appropriate. Several available flood routing models were considered for application along the study reach including the following: SSARR (17), HEC Unsteady Flow Profiles (19), NWS DWOPER (4), and FLOW 2D (15,18). A preliminary screening of available models lead to a detailed comparison of the NWS DWOPER model and the FLOW 2D

STUDY AREA MAP

Figure 1


Nashwaak Bridge
Taymouth
Nashwaak
01AK006
01AL002
River
Nashwaaksis
Burtts Corner
Keswick
River
Keswick
01AK005
Stream
River
Springhill
01AK003
Marysville
01AK004
Fredericton
PM Bridge
Mactaquac Dam
Saint John
Portobello
Upper Maugerville
Lincoln
Baker Brook
Baker Brook
Rusagonis
Maugerville
01AO001
01AO002
Oromocto
River
French Lake
01AM002
01AM001
Fredericton Junction
Oromocto
River
01AO009
Indian Lake
McGowans Corner
01AO005
01AO006
01AO007
French Lake
Maquapit Lake
01AO008
Lakeville Corner
Sheffield
Burton
Upper Gagetown
Little River
Newcastle
Newcastle Creek
Newcastle Bridge
01AO003
01AN001
01AN002
Chipman
Grand Lake
Trans Canada Highway
01AP002
Jemseg
01AO004
Jemseg
Gagetown
Lake
Washademoak
Saint John
Queenstown
Hampstead
River
Evandale
SECTION GAUGE AT EVANDALE
01AP003
Oak Point
Belleisle Bay
Norton
01AP001
Hampton
River
Nerepis
01AP006
Long Reach
Kennebecasis Bay
Rothesay
Westfield
01AP005
Saint John
SECTION GAUGE NEAR INDIANTOWN
PUGSLEY WHARF TIDAL GAUGE
N
LEGEND
▲ Manual Seasonal Water Level WSC
■ Continuous Flow WSC
● Continuous Level WSC
○ Additional Stations (Manual Water Level)
0 5 10 15 kilometres

model. The Dynamic Wave Operational Model, DWOPER, was developed by the Hydrologic Research Laboratory of the United States National Weather Serive. FLOW 2D is a proprietory model developed by Resource Analysis Inc.

A summary of the model comparison is given in Table 1. Additional details of the model comparisons including a discussion of preliminary simulation results are described elsewhere (12). This comparison resulted in the selection of the DWOPER model for application in this investigation. The following discussion pertains only to the DWOPER application.

MODEL DESCRIPTION

OUTLINE OF THEORY AND SOLUTION TECHNIQUE

The basis for DWOPER is a finite difference solution of the one-dimensional equations of unsteady flow. In a simplified form, these equations can be written as follows:

Continuity: $$\frac{\partial Q}{\partial x} + \frac{\partial (A + Ao)}{\partial t} - q = 0 \qquad (1)$$

Momentum: $$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \frac{(Q^2)}{(A)} + gA \left(\frac{\partial h}{\partial x} + S_f\right) - qV_x = 0 \qquad (2)$$

where:
h = water surface elevation
g = acceleration due to gravity
A = cross-sectional area of flow
Ao = off channel cross-sectional area
Q = discharge
x = distance in the direction of flow
t = time
S_f = friction slope (by Manning equation)
q = lateral inflow/outflow along the river
V_x = velocity of lateral inflow/outflow in the direction of channel flow

The derivation of these equations in their conservation form is given by Fread (3). These nonlinear partial differential equations are solved in DWOPER by a method referred to as the "weighted four-point" implicit scheme (5). Compared to other implicit finite difference techniques, this solution scheme has two desirable properties in that unequal distance steps can be used, and the stability-convergence characteristics can be controlled. In this technique, the continuous x-t region in which solutions of h and Q are sought is represented by a rectangular net of discrete points at equal or unequal intervals of time and space. Each point is identified by a subscript (i) which designates the distance (x) position, and a superscript (j) for the time position. The time derivatives are approximated by:

Table 1 **COMPARISON OF FLOW 2D AND DWOPER MODELS**

	FLOW 2D	DWOPER NWS
Core Requirements IBM	400^k	330^k
Cost	1^c/x-section/time step	2^c/x-section/time step
Equation of Motion	Assumes water surface slope equal to frictional slope	Incorporates terms of space and time derivatives in the equation
Numerical Solution	Identifies influence of boundary in X-Y plane and solution advances by checking continuity at each time step	Solution is identified in X-t plane where equations of motion and continuity are solved simultaneously
Accuracy	Acceptable tolerance between successive iteration is applied to stage only without model modifications.	Acceptable tolerance between successive iteration may be applied to both stage and discharge
Cross-section	Identified as control units between cells. Detached no. of prismatic cross-sections make the description of river discontinuous	Continuous description of the river where each intermediate cross-section is common between two adjacent elemental reaches
Manning's 'n'	Cannot account for variable Manning's 'n' with discharge or stage	Can incorporate variable Manning's 'n' with stage or discharge

	FLOW 2D	DWOPER NWS
Time Step	Variable time step. Program chooses its time step internally according to the requirement specified by the tolerance input	Time step and tolerance have to be input into the model
Calibration	Externally by trial and error	Automatic calibration routine is available as a part of the model
System-Specific Changes Required	To incorporate submerged weir, capability in the frame of the existing model	Weir-flow bifurcation could readily be incorporated as modifications were already underway by model developer
Reversing Falls	Cannot simulate flow through Reversing Falls	Can simulate flow through Reversing Falls
Forecast Mode	Not developed for operational flood forecasting	Developed for operational flood forecasting
Data Handling	Somewhat harder to incorporate new data	Easier to incorporate new data
Model Availability and Future Changes	Proprietory with development through private efforts	Readily available with continuous developed by a government agency

$$\frac{\partial K}{\partial t} \simeq (K_i^{j+1} + K_{i+1}^{j+1} - K_i^j - K_{i+1}^j) / 2\Delta t \qquad (3)$$

where K represents any variable. The spatial derivatives are approximated by a finite difference quotient positioned between two adjacent time lines according to weighting factors Θ and 1-Θ:

$$\frac{\partial K}{\partial x} \simeq \Theta(K_{i+1}^{j+1} - K_i^{j+1}) / \Delta x + (1-\Theta) (K_{i+1}^j - K_i^j) / \Delta x \qquad (4)$$

When Θ equals 1.0, a fully implicit scheme is formed, whereas if Θ equals 0.5, a box scheme results. The influence of the Θ weighting factor on the stability and convergence properties has been presented by the developer of the model(5). It has been found that accuracy decreases as θ departs from 0.5 and approaches 1.0. As recommended for most practical applications, a value of 0.55 has been used for Θ in this study in order to minimize loss of accuracy while avoiding weak instability as reported when Θ of 0.5 is used.

For each time step, the implicit numerical solution technique is applied to solve equations 1 and 2 simultaneously at all computational points in the river system. The resulting system of 2N nonlinear equations (N is the total number of cross-sections) is solved at each time step by the Newton-Raphson method which is an iterative procedure. In the Newton-Raphson method, a matrix of 2N x 2N linear equations is generated. The system's banded coefficient matrix is efficiently solved by a modified Gaussian elimination algorithm developed by Fread(5). As an example of the solution efficiency, the model of the lower Saint John typically converged after 1 to 2 iterations.

OUTLINE OF MODEL CAPABILITIES

Some important features of the DWOPER model utilized in its application to the lower Saint John River are briefly summarized as follows:

"Initial conditions" - Estimated stages and discharges required for initiating the simulation at each cross-section can be read in, or can be saved from a previous DWOPER simulation. An option is also available by which initial conditions can be computed by the model in a steady flow calculation.

"Boundary conditions" - Known stage or discharge hydrographs can be used as upstream and downstream boundary conditions. The downstream boundary can also be specified as a single-valued or loop rating curve, or as a weir.

"Cross-sections" - Irregular cross-sections located at unequal distances can be specified as pairs of top widths and elevations. Low flow areas can be used to describe the cross-section below the minimum elevation read in.

"Off-channel storage" - Areas in which the flow velocity in the x-direction is considered negligible can be specified as storage areas.

"Roughness coefficients" - Values of Manning's n can be specified as functions of either stage or discharge for each reach.

"Inflows" - Tributary inflows can be incorporated according to the lateral inflow term of equations 1 and 2. Tributaries can also be explicitly modelled in a manner similar to the main river, and, therefore, flows and stages are computed at tributary cross-sections. Usually, this is done when dynamic effects in the tributaries are considered important, as in the case of the Oromocto River and Portobello Stream. When only the effect of the tributary's flow on the main river system is modelled, the tributary hydrograph is input as lateral inflow, without the need for tributary cross-sections.

"Weir-flow bifurcation" - Flow transfer can be simulated to or from another channel which connects back into the main channel at some point downstream from the bifurcation. This was important in simulating flow over the Trans Canada Highway to Portobello Creek and adjacent storage areas.

"Automatic calibration" - An optimization technique can be applied to sequential reaches of a river system. The Manning's roughness coefficients are adjusted automatically until a minimum Root Mean Square (RMS) error for the reach is found.

The utilization of the above features is described in the next section. Other features of the model, not used in the Saint John River study, include provision for local expansion and contraction losses, the effects of wind resistance, target pool elevations for locks and dams and the capability to allow for flow diversion. In addition, several data management and display options are available for use of the model in the flood forecast mode.

PREPARATION OF DWOPER MODEL OF LOWER SAINT JOHN RIVER

Data Base

The most time-consuming activity of this study was the collection and preparation of a consistent data base. Existing data were used as much as possible in defining the channel and off-line storage characteristics. The sources of information included the following:

- 1:50,000 topographic mapping
- hydrographic charts
- flood hazard mapping in the Fredericton area (pilot project, 1975 (11))
- cross-sections provided by Professor D. I. Bray of the University of New Brunswick
- cross-sections provided by the New Brunswick Department of Transportation
- stage-area and stage-storage curves determined by Fisheries and Environment Canada (9)
- flow and water level data provided by the Water Survey of Canada, Environment New Brunswick and the New Brunswick Electric Power Commission.

Field surveys were conducted in order to augment the following parts of the data base:

- cross-section data in the study reach upstream from Fredericton (no hydrographic charts were available in this reach)
- cross-section data at the mouths of the Oromocto and Jemseg Rivers
- elevation data in the Portobello Stream storage area north of the Trans Canada Highway
- longitudinal profile of the Trans Canada Highway in the Maugerville-Sheffield areas
- flow measurements upstream from the Reversing Falls.

In order to minimize computer costs, "average" cross-sections were derived for uniform reaches from the measured cross-sections. This cross-section averaging was accomplished using a simple data processing program in which the weighting factor is the distance between the cross-sections. In this manner, a total of more than 170 cross-sections from McKinley Ferry to Pugsley Wharf, including the tributaries, were reduced to 56 cross-sections which were input to the model.

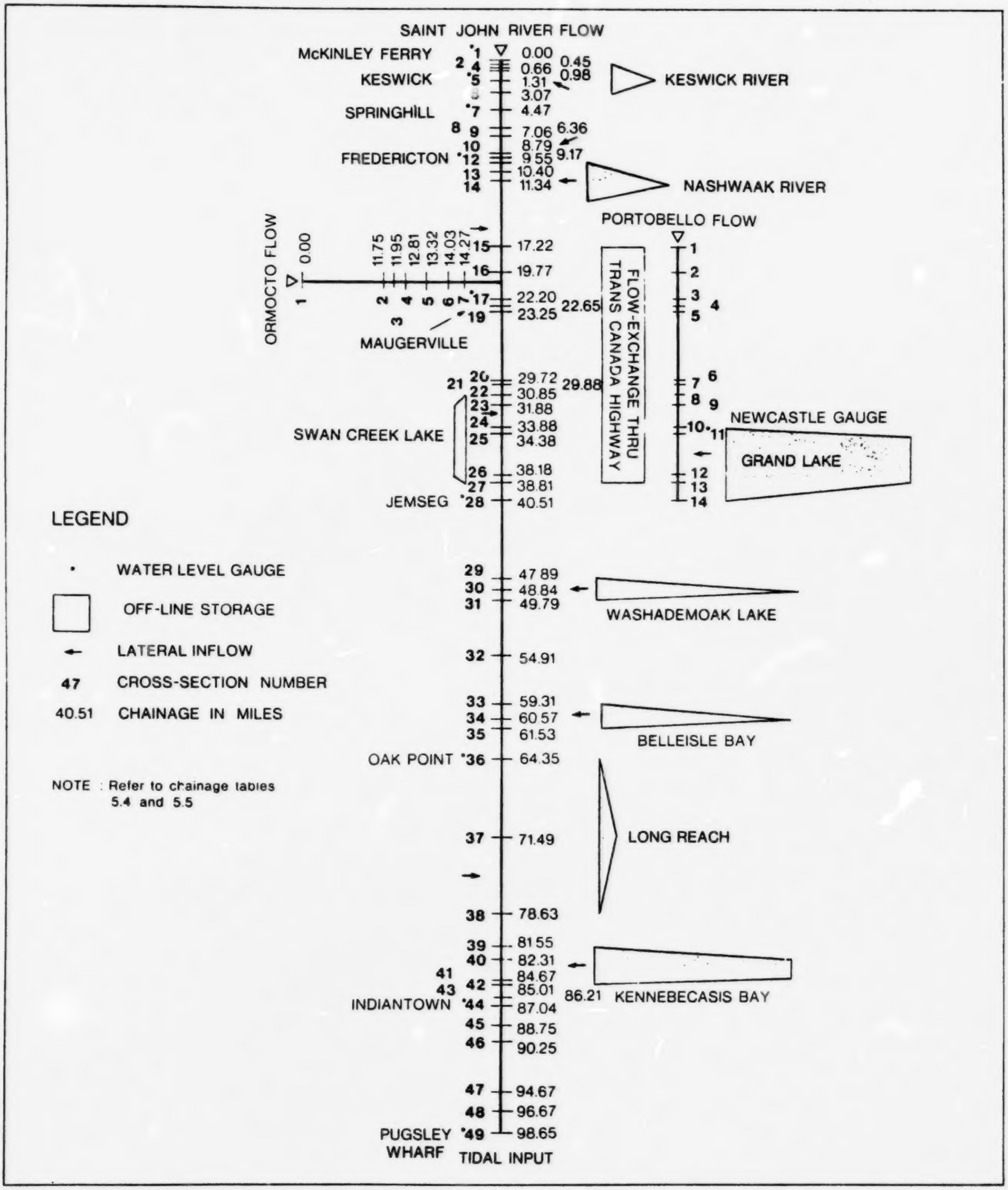
System Schematization

The mathematical representation of the lower Saint John River by the DWOPER model could be accomplished by any one of several system representations. Consideration of preliminary simulations and study constraints lead to the adoption of the system schematic shown as Figure 2. The main components of the model are discussed in the following paragraphs together with an attempt to highlight some of the previously mentioned model capabilities.

The upstream boundary of the model is the discharge hydrograph measured at the Mactaquac gauge. Channel routing is the predominant hydraulic feature of the reach from McKinley Ferry to Fredericton, with relatively small off-line storages included at the mouth of the Keswick and Nashwaak Rivers. Intermediate drainage areas are accounted for by including lateral inflows.

SCHEMATIC OF DWOPER MODEL OF THE LOWER SAINT JOHN RIVER

Figure 2


SAINT JOHN RIVER FLOW
McKINLEY FERRY
KESWICK
SPRINGHILL
FREDERICTON
KESWICK RIVER
NASHWAAK RIVER
PORTOBELLO FLOW
ORMOCTO FLOW
MAUGERVILLE
FLOW-EXCHANGE THRU TRANS CANADA HIGHWAY
SWAN CREEK LAKE
NEWCASTLE GAUGE
GRAND LAKE
JEMSEG
WASHADEMOAK LAKE
BELLEISLE BAY
OAK POINT
LONG REACH
KENNEBECASIS BAY
INDIANTOWN
PUGSLEY WHARF
TIDAL INPUT
0.00
0.45
0.66
0.98
1.31
3.07
4.47
6.36
7.06
8.79
9.17
9.55
10.40
11.34
11.75
11.95
12.81
13.32
14.03
14.27
17.22
19.77
22.20
22.65
23.25
29.72
29.88
30.85
31.88
33.88
34.38
38.18
38.81
40.51
47.89
48.84
49.79
54.91
59.31
60.57
61.53
64.35
71.49
78.63
81.55
82.31
84.67
85.01
86.21
87.04
88.75
90.25
94.67
96.67
98.65
LEGEND
WATER LEVEL GAUGE
OFF-LINE STORAGE
LATERAL INFLOW
47 CROSS-SECTION NUMBER
40.51 CHAINAGE IN MILES
NOTE : Refer to chainage tables 5.4 and 5.5

The reach from Fredericton to Lower Jemseg is hydraulically more complex. Preliminary model simulations demonstrated that flow exchange with the Grand Lake - Portobello Stream storage area was difficult to account for as an off-line storage area due to overtopping of the Trans Canada Highway at high stages. A general modification to the original DWOPER model was, therefore, undertaken with the model developer, Dr. D. L. Fread. This modification allows flow transfer from one or more cross-sections in the main river to or from an associated set of cross-sections along Portobello Stream as indicated on Figure 2. The rate of inflow/outflow is a function of water levels at the related cross-sections in the current time step as governed by the weir flow equation.

Flow transfer to the Grand Lake off-line storage area is, therefore, simulated either as a flow reversal up the Jemseg River (modelled as a continuation of Portobello Stream) or by flow across the Trans Canada Highway or by a combination of both mechanisms. These flow transfer mechanisms have been observed during historical flood events (9).

On the basis of preliminary simulations it was concluded that backwater effects from the Reversing Falls as far upstream as Jemseg precluded the development of a general rating curve for the downstream boundary of the study reach at Lower Jemseg. Therefore, it was found to be necessary to model the reach from Lower Jemseg to Pugsley Wharf in order to incorporate the hydraulic effects of the Reversing Falls.

The model of the reach from Jemseg to Indiantown accounts for several off-line storage areas and tributary inflows as shown on Figure 2. The relations for distributing tributary inflows, by pro-rating measured flow values according to drainage area, were adopted according to the results of a previous model study of the lower Saint John River (16).

The reach from Indiantown to Pugsley Wharf incorporates the influence of the Reversing Falls and channel constriction effects in this area. The downstream boundary of the model is input as the tidal stage hydrograph at Pugsley Wharf. It was found necessary to adopt a two-hour computational time step in order to adequately simulate the effects of the tide.

MODEL CALIBRATION AND VERIFICATION

An important part of the model development involved the undertaking of detailed calibration and verification simulations. The main calibration parameter is the Manning roughness coefficient - n. Values for n were determined with the aid of the model's automatic calibration feature (6), as well as by trial and error simulations. The following flood events were used for calibration purposes:

- April 21 to May 5, 1973
- April 27 to May 7, 1974
- April 19 to April 30, 1977

Calibration attempts revealed that better simulations could be achieved by varying n with stage rather than flow along the lower Saint John River.

Based on the calculated n values for each reach for the three calibration runs, final average n values were adopted and these values were used in all subsequent simulations. An example comparing observed and simulated stage hydrographs for the 1973 flood is given on Figure 3. Subsequently, the spring flood of 1978, from April 26 to May 15, was simulated in order to verify the model.

The Root Mean Square (RMS) error and the error in peak levels are given for all gauge locations and all four flood events in Table 2. These comparisons indicate that the calibrated model gives a good simulation accuracy throughout the river system. Slightly better simulation accuracies were obtained along the upstream study reach where additional field measurements were taken during the study. The level of accuracy is comparable to that attained by other DWOPER simulations such as those carried out along the Mississippi River and its tributaries (4).

Comparison of simulated flows to those measured at Indiantown during the 1978 spring flood show agreement to within ±10%. A similar comparison for the 1977 flood agrees to within ±10 to 20%, but the flow measurements were less extensive and possibly less accurate than those taken in 1978.

On the basis of these stage and discharge hydrograph comparisons, it was concluded that the DWOPER model provided a good simulation of historical flood events. However, further improvements to the model performance might be achieved by a better representation of the Reversing Falls. Extensive flow measurements at the Reversing Falls over a wide range of water levels at Indiantown and Pugsley Wharf could lead to the formulation of a rating curve for the Falls. Such a rating curve might be an improvement on the present stage hydrograph at Pugsley Wharf as the downstream boundary. However, this type of rating curve could not be handled by the existing version of the DWOPER model, and, therefore, would require a general model modification.

Another facet of the model with potential for improvement is the estimation of tributary inflow. At present, only one-third of the drainage area downstream of McKinley Ferry is gauged.

Figure 3

COMPARISON OF OBSERVED AND SIMULATED STAGE HYDROGRAPHS FOR 1973 FLOOD

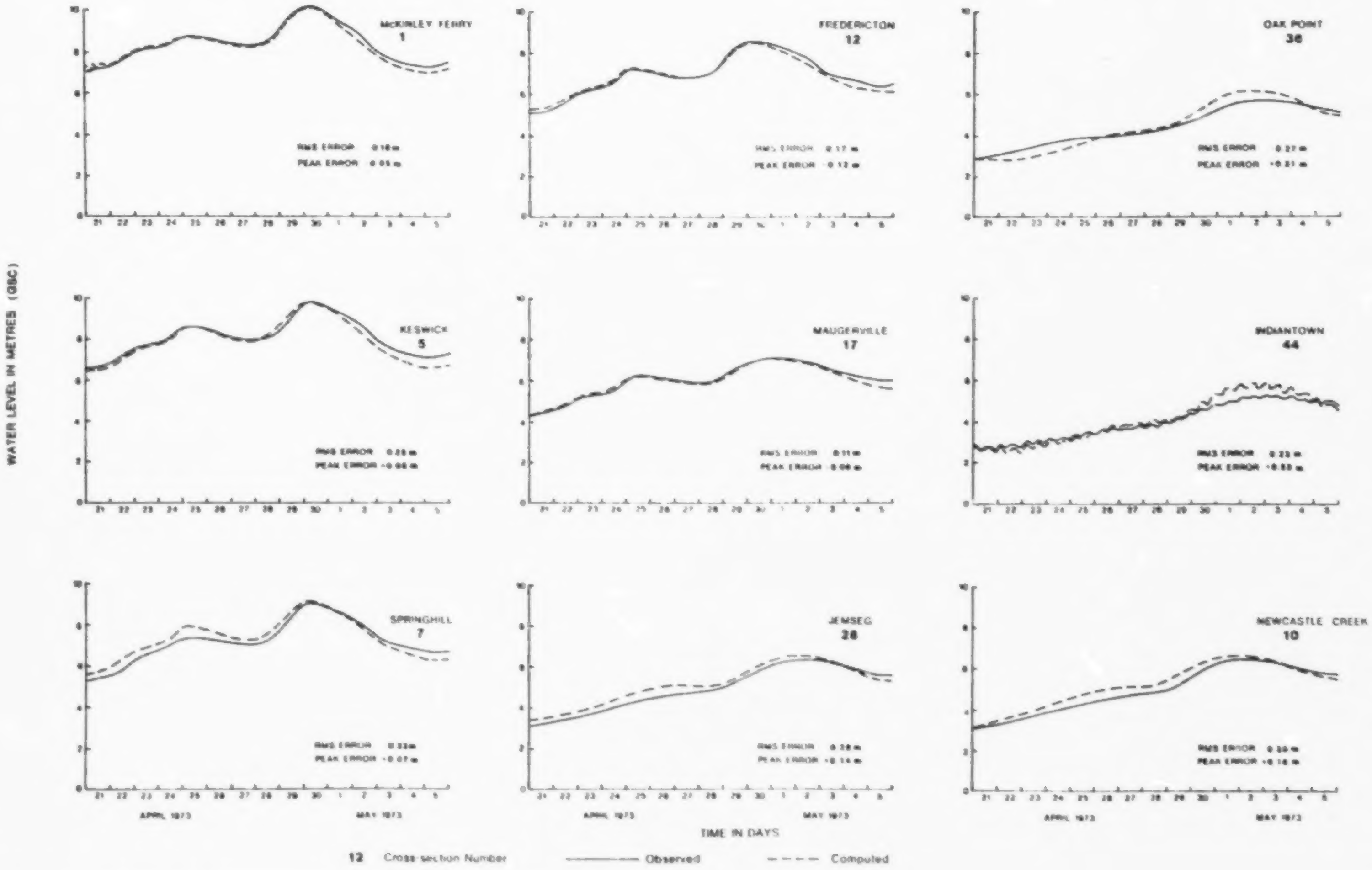


TABLE 2 : SUMMARY OF PEAK* AND RMS ERRORS (IN METERS) FOR CALIBRATED DWOPER MODEL SIMULATIONS OF SPRING FLOODS

	1973		1974		1977		1978	
LOCATION	PEAK	RMS	PEAK	RMS	PEAK	RMS	PEAK	RMS
McKinley Ferry	-0.05	0.16	-0.14	0.18	-0.15	0.11	-0.02	0.04
Keswick	+0.08	0.25	N/A	N/A	N/A	N/A	-0.10	0.21
Springhill	+0.07	0.33	N/A	N/A	+0.11	0.26	+0.59	0.49
Fredericton	-0.12	0.17	-0.07	0.13	-0.20	0.20	-0.14	0.20
Princess M Bridge	N/A	N/A	N/A	N/A	+0.09	0.15	N/A	N/A
Baker Brook	N/A	N/A	N/A	N/A	-0.33	0.25	-0.19	0.28
Maugerville	-0.06	0.11	-0.19	0.13	-0.16	0.17	-0.16	0.22
Jemseg	+0.14	0.29	+0.22	0.21	-0.02	0.12	-0.06	0.16
Oak Point	+0.31	0.27	+0.21	0.16	-0.05	0.08	-0.08	0.21
Indiantown	+0.53	0.23	+0.11	0.14	-0.23	0.17	-0.18	0.28
Oromocto at French Lake	N/A	N/A	-0.02	0.17	-0.18	0.16	N/A	N/A
Oromocto Mouth	N/A	N/A	N/A	N/A	-0.18	0.22	N/A	N/A
Portobello at Burns Bridge	N/A	N/A	+0.61	0.42	+0.05	0.29	N/A	N/A
French Lake above Lakeville Corner Bridge	N/A	N/A	+0.32	0.24	N/A	N/A	N/A	N/A
Maquapit Lake below Lakeville Corner Bridge	N/A	N/A	+0.42	0.25	N/A	N/A	N/A	N/A
Newcastle Creek	+0.16	0.30	+0.32	0.25	+0.01	0.15	-0.06	0.12
Averages**	0.17	0.23	0.24	0.21	0.14	0.18	0.16	0.22

NOTE: N/A = No measurements available

* Error between observed and simulated peak water levels

** Average of absolute values of peak errors

1:20 AND 1:100 YEAR RECURRENCE INTERVAL FLOOD PROFILES

As stated earlier, a study objective was to determine the flood profiles associated with the 1:20 and 1:100 year recurrence interval floods. Flows determined by the recent New Brunswick Regional Frequency Analysis (1) were utilized.

The hydrograph characteristics and the tidal input at Pugsley Wharf were determined from an analysis of historical records. A sensitivity analysis determined that statistical error in hydrologic data resulted in the most significant changes to the 1:20 and 1:100 year flood profile. The mean profiles estimated for the ±95% confidence limits are shown on Figure 4. The maximum flood of record (1973) is also shown for comparison.

CONCLUSIONS

The use of the DWOPER unsteady flood routing model resulted in an acceptable simulation accuracy for flood levels along the lower Saint John River. As a result of the successful calibration and verification of the model, it was possible to simulate the 1:20 and 1:100 year flood profiles along the study reach. When fully implemented in the flood forecast mode, it is expected that the model will forecast levels more accurately than the technique currently in use for flood forecasting along the lower Saint John River.

It is felt that additional refinements to the unsteady flood routing model of the lower Saint John River could only be undertaken after the collection of additional field data. This would include additional gauging of tributary inflows and implementation of a program to collect continuous flow measurements through the Reversing Falls. Future model refinements would then include adoption of a rating curve at the Reversing Falls as the downstream boundary condition.

ACKNOWLEDGEMENT

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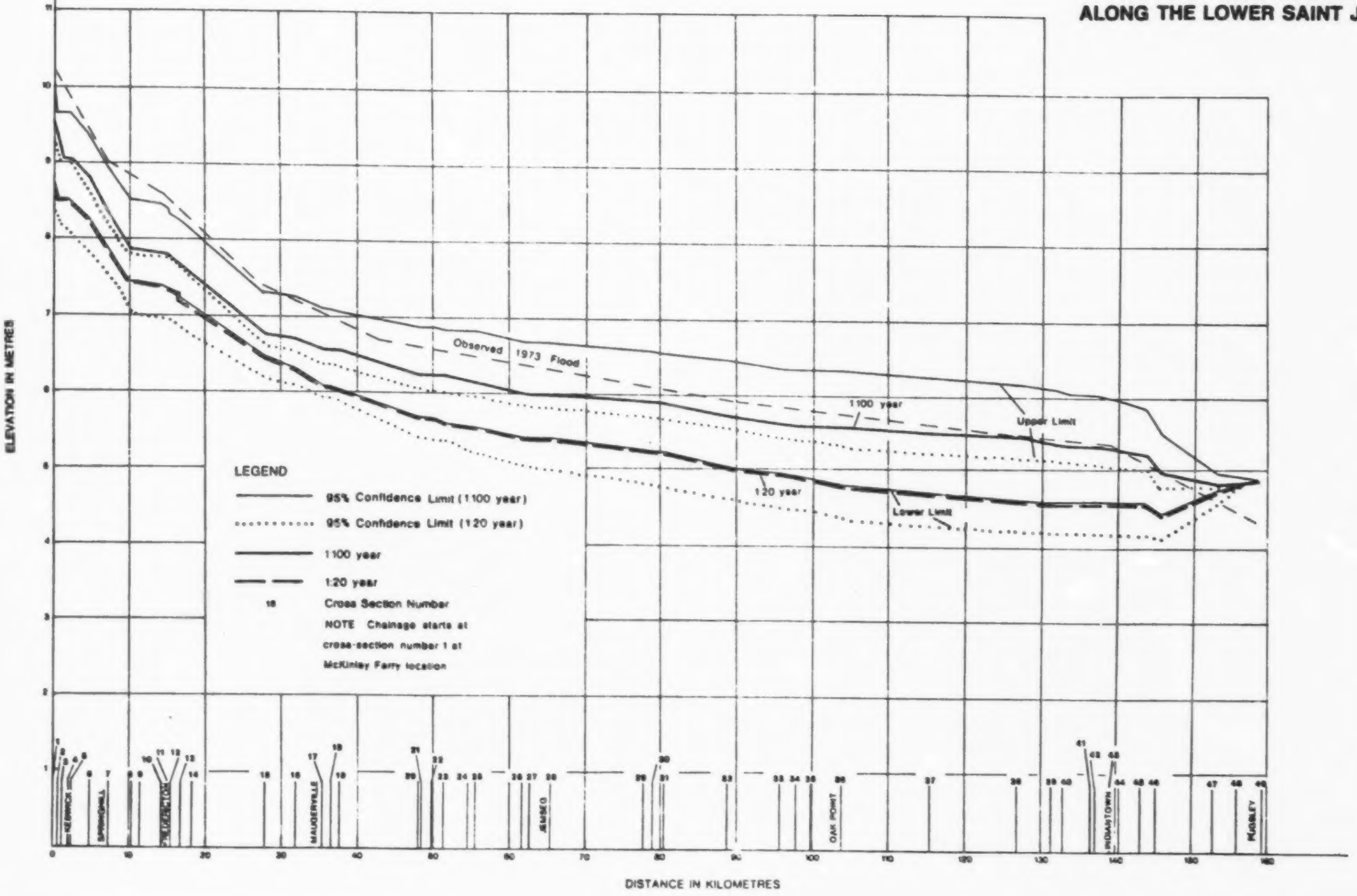
Valuable assistance throughout this study was provided by members of the Hydro-Technical Sub-Committee, as well as staff from Environment New Brunswick, the Inland Waters Directorate, Resource Analysis Incorporated, and the New Brunswick Electric Power Commission. In addition, we would like to thank Dr. D. I. Bray, of the University of New Brunswick, Dr. P. E. Wisner, of the University of Ottawa, and especially Dr. D. L. Fread, of the U.S. National Weather Service.

Figure 4
PROFILES FOR
1:20 AND 1:100 YEAR LEVELS
ALONG THE LOWER SAINT JOHN



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APPLICATION OF FINITE ELEMENT METHOD IN RIVER NETWORK HYDRAULICS

by

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Abstract

This paper describes a finite element model of the unsteady flow regime in the open-channel networks. Mathematically, one-dimensional unsteady flow hydraulics of open channels is described by a pair of nonlinear, partial differential equations of hyperbolic type and is solved by adopting linear elements.

A Predictor-Corrector method for solving the nonlinear equations is utilized. Simulation capabilities of the computerized model were investigated in determining the steady-state profile for the St. Lawrence River. Limited scale sensitivity analysis was conducted to study the influence of the variation of the weighting parameter.

A comparison of the model output with the observed events and/or calculated values indicates the suitability of the model from the viewpoints of simulation capabilities, adoption of higher time steps, conservation of mass, flexibility of operation and handling of complex configuration of networks.

Introduction

The Saint-Venant equations approximately describe unsteady flow hydraulics of rigid open channels. Numerous numerical techniques have been developed to study the unsteady flow phenomenon of rivers, channels and open-channel networks. These solutions can basically be divided as finite difference, finite element, and characteristic methods. Amien et al. (1970), Contractor (1972, Quinn and Hagman (1977), Strelkoff (1970), Viessmann (1972). and Moin (1974) studied flood routing along open channels by finite difference methods; Larson et al. (1971) and Amien and Fang (1969) utilized the method of characteristics for solving the unsteady flow equations. Cooley and Moin (1976) and Machemehl and Gopalakrishma (1977) solved the Saint-Venant equations by the finite element method.

One drawback of most of the one-dimensional open-channel models is that the lateral flows are handled only as distributed flow along the channels. This precludes the possibility of studying back-water conditions at channel junctions, the study of flow past islands and the hydraulics of open-channel networks. A review of literature revealed several studies some of which date back to 1957. Stoker (1957) was first to study the hydraulics of open-channel junctions; he studied the classical problem of Ohio-Mississippi River Junction. Larson et al. (1970) reported findings of a laboratory scale model. Both the above studies utilized the method of characteristics to solve the problem. Quinn and Hagman (1977) and Muir (1975) developed implicit finite difference models. Machemehl and Gopalakrishna (1977) developed a finite element model describing the hydraulics of open-channel networks.

This paper describes a computerized finite element model based on the Galerkin method. The finite element solution of the Saint-Venant equations is the same as the one developed by Cooley and Moin (1976) and is extended to include channel networks.

Unsteady Flow Equations

In the present work, the Saint-Venant equations describing conservation of momentum and mass in rigid open channels are given as follows:

$$\frac{\partial Q}{\partial t} + \frac{\partial (VQ)}{\partial x} + gA\,\frac{\partial y}{\partial x} - gA(S_o - S_f) - u_x q = 0 \qquad (1)$$

and

$$T\,\frac{\partial y}{\partial t} + \frac{\partial Q}{\partial x} - q = 0 \qquad (2)$$

in which Q = volumetric discharge; x = length in the longitudinal direction; y = depth of flow; t = time; V = the average velocity of flow in a cross-section; T = the top width of cross-section or the rate of change of area with depth; S_o = the longitudinal slope of the channel bottom; $S_f = n^2 V|V|/2.21R^{4/3}$; n = Manning's roughness parameter; R = hydraulic radius; q = the lateral inflow per unit length of channel (negative for outflow); u_α = the average component of inflow velocity in the x direction; and g = the acceleration due to gravity.

Equations (1) and (2) are used for describing the hydraulics of all interior nodes (Fig. 1) for subcritical flow.

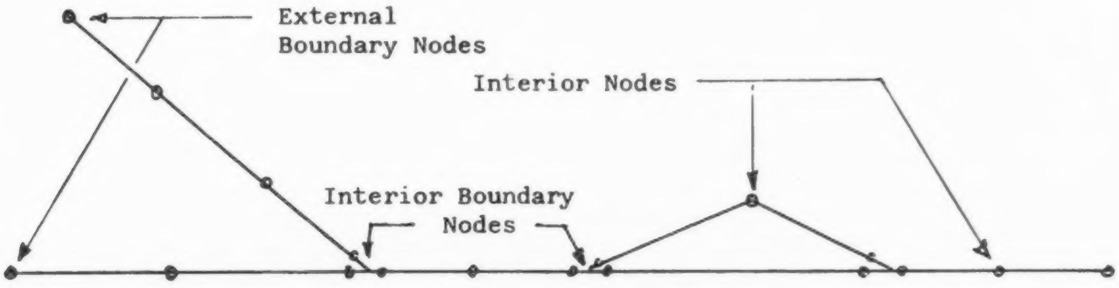


Fig. 1. Definition of Channel Network

Boundary Conditions

For subcritical flow, Strelkoff (1970) recommends using the forward and backward characteristic equations:

$$\frac{\partial Q}{\partial t} + Q\,\frac{\partial V}{\partial x} \pm C\,\frac{\partial Q}{\partial x} - T\,(V \pm C)\,\frac{\partial y}{\partial t} + gA\,\frac{\partial y}{\partial x}$$

$$- gA(S_o - S_t) - u_x + (V \pm C)q = 0 \qquad (3)$$

Where: $C = \sqrt{gA/T}$

Equation (3) is used along with a relationship depending upon whether it is an internal or external boundary.

Interior Boundary Condtions: In the development of this model, internal boundaries are considered at junctions with tributaries, branching around islands, sudden expansion or contraction in the channel, flux discharges over wiers, etc.

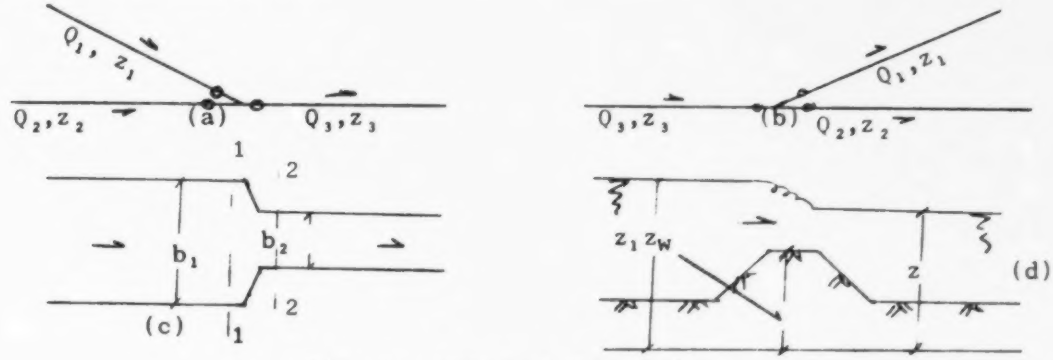


Fig. 2. Internal Boundaries

Even though all internal boundaries are essentially described by the compatibility conditions viz: continuity condition for the discharge and the compatibility of energy line across the transition, the following relations are used:

a) Open Channel Junctions (Fig. 2, (a) and (b)). The discharge in channel 3 is equated to the flows in channels 1 and by by the relation:

$$Q_1 + Q_2 = Q_3 \qquad (4)$$

The energy head at nodes 1, 2, and 3 is constant given as:

$$z_1 + \frac{V_1^2}{2g} = z_2 + \frac{V_2^2}{2g} = z_3 + \frac{V_3^2}{2g} \qquad (5)$$

where: z_1, z_2, z_3 = water surface elevation at nodes 1, 2, and 3.

b) Sudden change of the cross-section between two reaches (Fig. 2 (c)). The compatibility conditions are:

$$Q_1 = Q_2 \qquad (6)$$

and $$z_1 + \frac{V_1^2}{2g} = z_2 + \frac{V_2^2}{2g} \tag{7}$$

c) Discharge over weirs (Fig. 2 (d)). This requires:

$$Q_1 = Q_2$$

and $$f(z_1, z_w, z_2) = 0 \tag{8}$$

Equation (8) describes stage relationship for various z_1 and z_2 values.

Exterior Boundary Conditions: The exterior boundary conditions are usually of three kinds. All three can be described by:

$$Ay + BQ = C \tag{9}$$

where A, B, and C are known functions. For upper and lower boundaries, A is0 and B = 1 for discharge hydrograph and B is zero and A = 1 for stage hydrograph. For downstream boundary A, B, and C express segments of a rating curve. In case the rating curve is nonlinear, it is linearized using Taylor expansion.

Initial Conditions

Initial conditions employed are those of steady-state flow. In the model development, it is considered that the values of discharge Q and the depth of flow y are known initially.

Finite Element Method

The numerical solution of the mass continuity and momentum equation is obtained by the finite element method. The finite element solution based on the Galerkin formulation is used herein and follows from the earlier work of Cooley and Moin (1976). Background information on the development of the finite element method may be found in standard text books of Pinder and Gray (1977), Huebner (1975), and Connor and Brebbia (1976).

The continuum describing the physical channel network is approximated by using linear elements interconnected at the nodes. The dependent variables, depth of flow (y) and discharge (Q), are interpolated from the nodal values to describe continuous approximations for the entire network.

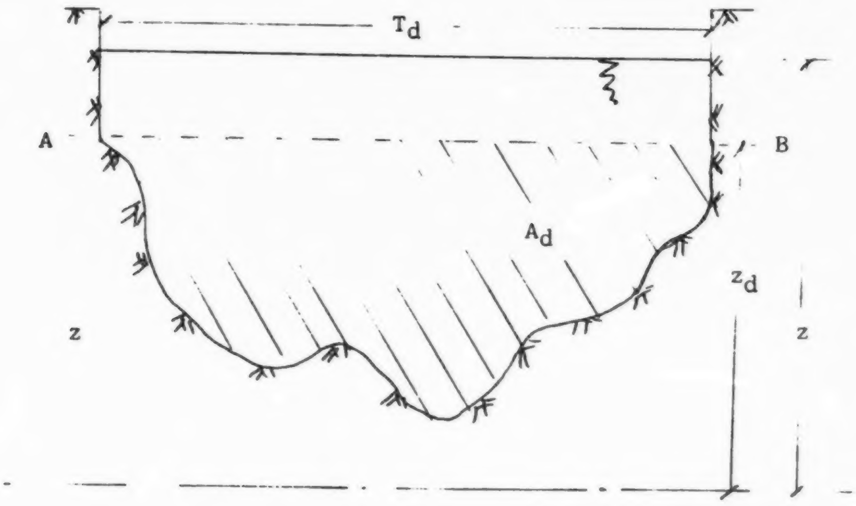
Linear approximating functions are used in this study. This requires that the dependent variables Q and y be defined as functions of x, i.e., Q = Q(x) and y = y(x). Once the element network and the basic functions for approximating are specified, the Galerkin method is applied to equations (1), (2), and (3). This involves a) multiplying the equations by the weighting functions of Q(x) and y(x), b) substitution of the functions in the equations, c) integration of the equations thus obtained over each element, which leads to a set of ordinary differential equations in time involving the unknowns and d) numerical integration of the equations over time. For details of the methodology, the reader is referred to the paper by Cooley and Moin (1976).

Application of the Finite Element Model

In this exercise, the model was used to obtain a steady-state profile in the St. Lawrence River between Kingston and Cornwall for a set of given discharges at Cornwall and a fixed Lake Ontario elevation.

In an exercise for the Lake Erie/Lake Ontario Regulation Study, hydraulic data was obtained from the U.S. Army Corps of Engineers. The same data is used in this exercise. The reach of about 101.4 miles between Kingston and Cornwall is divided up into 31 elements. Each element was assigned 2 nodes, resulting in a total of 62 nodes. There are 3 external boundary nodes; these are node numbers 1, 11, and 62. There are a total of 22 internal boundaries of which 14 are formed due to channel junctions, and the other 8 are created due to sudden change in cross-section between any two reaches. The schematic of the network is presented in Fig. 5. The channel for the first 15 reaches is predominantly an extension of Lake Ontario. These reaches are marked by large irregular cross-section area and very wide top widths. The only exception was the connecting navigation channels like element 2, which are well defined narrow channels of almost rectangular cross-section.

The length of the elements were variable with an average length of 17,170 feet. The longest element was 75,000 feet long and the shortest 5,930 feet. (One element of 100 feet was used to break up a junction of two inflow and two outflow reaches.) Manning's roughness coefficient varied from 0.022 to 0.041. A physical description of the St. Lawrence River is presented in Table 1. A typical cross-section of the St. Lawrence River is shown in Figure 3.



$$\text{Area } A = A_d + T_d(z-z_d)$$

$$\text{Wetted Perimeter } P = P_d + 2(z-z_d)$$

$$P_d = \text{Wetted Perimeter below AB}$$

Fig. 3. Typical Cross-Section of St. Lawrence River

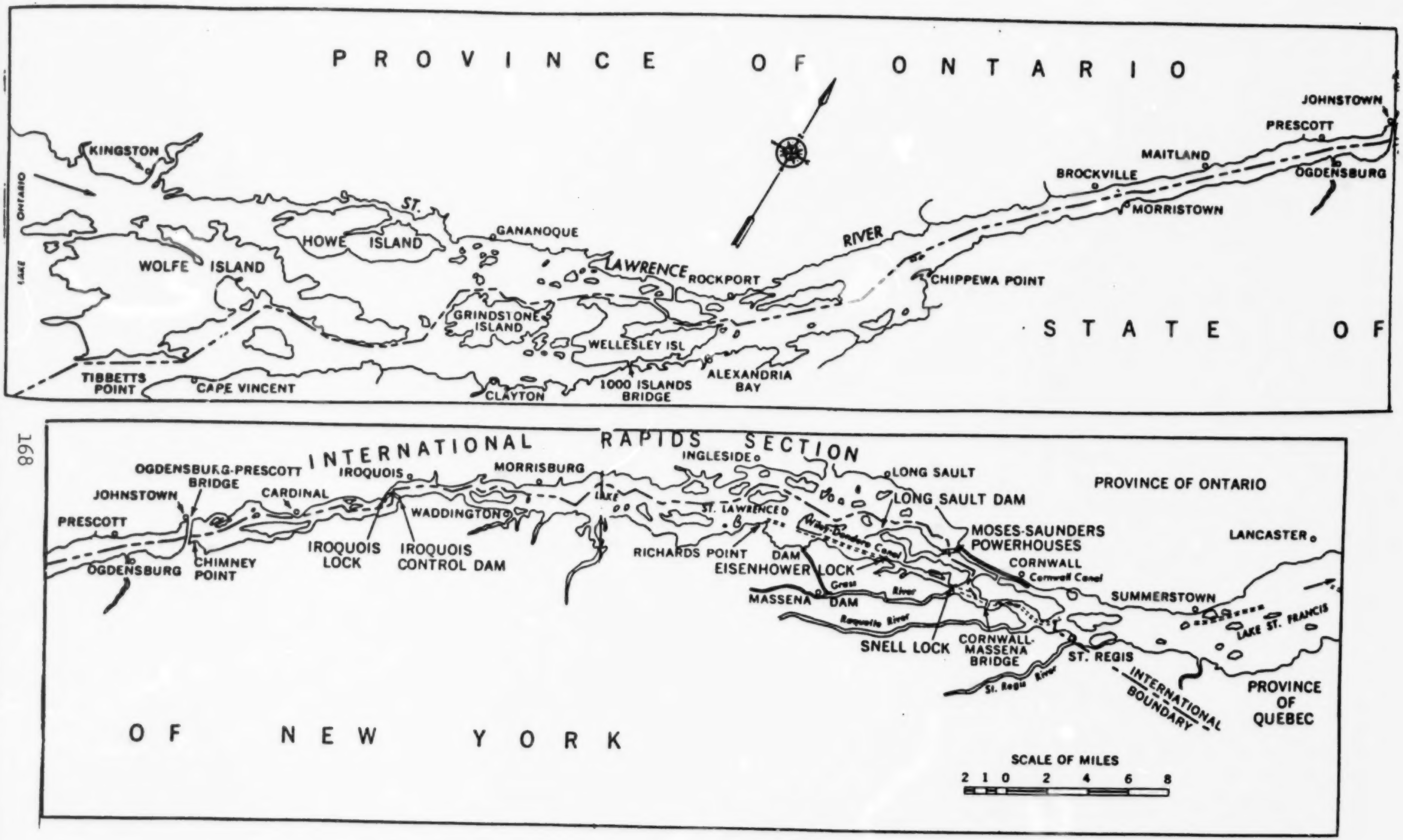

PROVINCE OF ONTARIO
STATE OF NEW YORK
LAKE ONTARIO
KINGSTON
WOLFE ISLAND
HOWE ISLAND
ST. LAWRENCE RIVER
GANANOQUE
GRINDSTONE ISLAND
WELLESLEY ISL
TIBBETTS POINT
CAPE VINCENT
CLAYTON
1000 ISLANDS BRIDGE
ALEXANDRIA BAY
ROCKPORT
CHIPPEWA POINT
BROCKVILLE
MORRISTOWN
MAITLAND
PRESCOTT
JOHNSTOWN
OGDENSBURG
INTERNATIONAL RAPIDS SECTION
OGDENSBURG-PRESCOTT BRIDGE
CHIMNEY POINT
CARDINAL
IROQUOIS
IROQUOIS LOCK
IROQUOIS CONTROL DAM
WADDINGTON
MORRISBURG
LAKE ST. LAWRENCE
INGLESIDE
RICHARDS POINT
DAM
EISENHOWER LOCK
Wiley-Dondero Canal
MASSENA
DAM
Grass River
Raquette River
LONG SAULT
LONG SAULT DAM
MOSES-SAUNDERS POWERHOUSES
SNELL LOCK
CORNWALL-MASSENA BRIDGE
CORNWALL
Cornwall Canal
ST. REGIS
St. Regis River
SUMMERSTOWN
LANCASTER
LAKE ST. FRANCIS
PROVINCE OF ONTARIO
PROVINCE OF QUEBEC
INTERNATIONAL BOUNDARY
SCALE OF MILES
2 1 0 2 4 6 8

Fig. 4. Area of Study
St. Lawrence River - Kingston to Cornwall

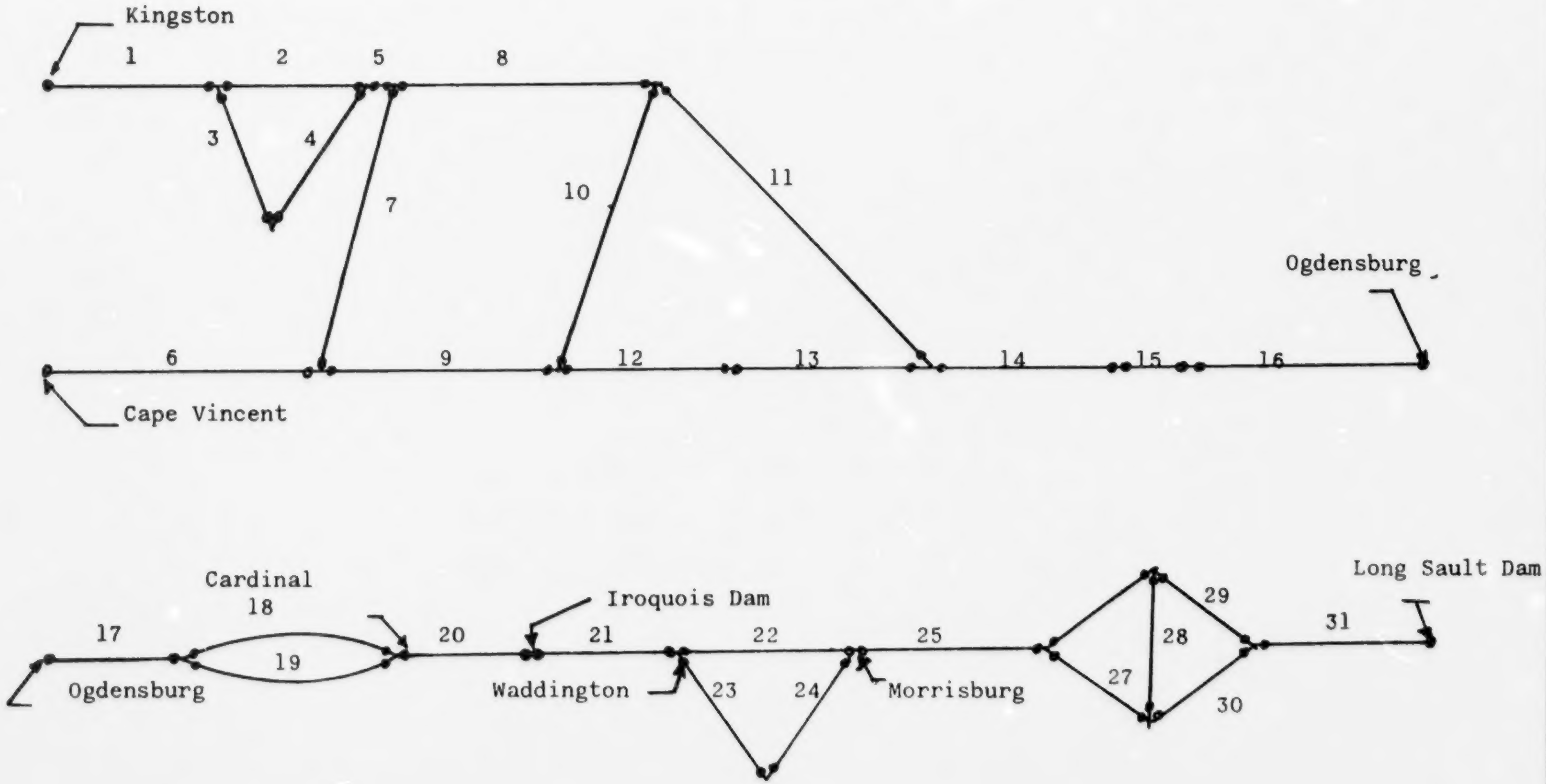

Kingston
1
2
5
8
3
4
7
10
11
Ogdensburg
6
9
12
13
14
15
16
Cape Vincent
Cardinal
18
Iroquois Dam
Long Sault Dam
17
19
20
21
22
25
29
28
31
27
30
Ogdensburg
Waddington
23
24
Morrisburg

Fig. 5. Schematic Plan of St. Lawrence River
Kingston to Cornwall

Table 1

Physical Description of St. Lawrence River
- Kingston to Cornwall -

Reach No.	Length (ft)	Manning's Coefficient	Average Area Below Datum (ft^2)	Netted Perimeter Below Datum (ft)	Top Width (ft)
1	35,000	0.022	366,800	6,536	6,422
2	62,800	0.035	45,100	1,579	1,520
3	28,130	0.027	264,250	4,155	3,977
4	27,200	0.022	669,100	13,495	13,400
5	100	0.035	187,100	6,508	6,450
6	68,700	0.022	356,500	6,629	6,520
7	10,500	0.030	62,000	1,926	1,860
8	22,200	0.040	345,500	7,005	6,905
9	28,100	0.035	187,100	6,508	6,450
10	16,600	0.031	60,400	1,590	1,510
11	50,800	0.033	195,600	3,728	3,620
12	15,000	0.036	255,450	4,635	4,550
13	49,100	0.034	148,800	1,983	1,820
14	75,000	0.026	260,900	7,669	7,600
15	15,000	0.027	123,700	3,315	3,240
16	65,900	0.026	182,200	4,325	4,240
17	17,200	0.033	110,500	3,800	3,740
18	28,100	0.034	58,500	1,615	1,540
19	30,000	0.034	27,600	1,083	1,030
20	23,100	0.026	87,800	2,685	2,620
21	16,900	0.028	98,300	2,724	2,650
22	17,300	0.026	58,000	2,046	1,988
23	9,750	0.030	43,800	1,597	1,540
24	9,650	0.030	76,300	1,639	1,540
25	5,250	0.041	179,300	4,845	4,780
26	20,600	0.041	86,400	1,680	1,570
27	11,900	0.041	150,000	2,945	2,840
28	5,930	0.041	76,800	1,630	1,530
29	13,100	0.041	169,400	3,364	3,260
30	19,700	0.041	60,600	1,130	1,010
31	15,900	0.041	232,500	6,858	6,790

FERNS was run to establish a steady-state profile in the river for a discharge of 262,000 cfs at Node 62 and a Lake Ontario elevation of 245.07 feet (Nodes 1 and 11). Initially, all the nodes were set at a horizontal pool level of 245.07 feet (same level as Lake Ontario) and a discharge of 1 cfs at all nodes. Theoretically, the flow should be zero, but the solution algorithm is so set up that a non-zero discharge is necessary. A flow of 262,000 cfs is then imposed on Node 62, and the model is allowed to run to a steady state.

In the second test run made in this exercise, a flow of 304,000 cfs was recorded at the Cornwall powerhouse with an observed Lake Ontario elevation of 245.56 feet. The computed and observed levels are presented in Table 2. As in the first exercise, runs were made with weighting parameters of 0.55, 0.67, 0.75, and 1.00. The computed steady-state profile in all the runs was practically the same. But, as the calculations are made for a steady-state profile, it cannot be inferred that one weighting parameter is superior to another. Another run was made utilizing the continuity equation along with the boundary conditions. This run, however, produced erroneous stages at the downstream boundary.

In this run, initially the stage at all the nodes was set at Lake Ontario's level of 245.56 feet with a nominal positive discharge of 1 cfs at all points. A flow of 304,000 cfs is then imposed at node 62, and the model is allowed the run to a steady-state profile; steady state is assumed when the flow and stage do not differ by less than 0.001 per cent. As in the first case, the computed and observed stages along the St. Lawrence River were found to match favorably.

The model was also tested for a third run with a flow of 277,000 cfs at the powerhouse and a Lake Ontario elevation of 243.72 feet. The computed and observed levels are presented in Table 2. The result of changing the weighting parameter was not different than what was observed in previous test cases. Also, as in previous cases, initially the stages along the river are set for a pool level and then allowed to adopt a steady state. In this run, the observed and the computed stages were found to be different. The reason for the differences was not investigated in detail due to the fact that the datums, stage measurements, etc., were not known. However, some light is cast in the following section.

Discussion on the Test Runs

In the first two runs, the observed and computed stages along the St. Lawrence River were found to match exceptionally well. At most of the points, the calculated stage was within 0.1 feet of the observed stage. The physical description of the river network is based on the survey undertaken by the U.S. Corps of Engineers and supplied to this Department for studying dredging alternatives for the Lake Erie/Lake Ontario Regulatory Works Subcommittee. The values for the Manning's roughness coefficient are based on some field work done earlier. The sensitivity of this parameter was not investigated.

It was also found in all three test cases that the flows do not converge to a single value. The explanation that can be given for this difference is that in the model linearity of all parameter variation is assumed, and the variables may be represented better by a nonlinear relation.

Table 2

Comparison of Computed and Observed Flow
- St. Lawrence River-Kingston to Cornwall -

Case 1. Outflow at Powerhouse 262,000 cfs, Lake Ontario level 245.07 feet.
Case 2. Outflow at Powerhouse 304,000 cfs, Lake Ontario level 245.56 feet.
Case 3. Outflow at Powerhouse 277,000 cfs, Lake Ontario level 243.72 feet.

No.	Station	Case 1		Case 2		Case 3	
		Computed	Observed	Computed	Observed	Computed	Observed
1	Kingston	245.07	245.07	245.56	245.56	243.72	243.72
2	Cape Vincent, N.Y.	245.07	245.07	245.56	245.56	243.72	243.72
3	Ogdensburg	244.39	244.36	244.70	244.61	242.89	-
4	Chimney Point	243.95	-	244.13	-	242.33	242.29
5	Cardinal	242.83	242.80	242.65	242.64	240.87	240.31
6	Iroquois HW	242.35	242.27	241.98	241.96	240.21	239.47
7	Iroquois TW	242.16	242.19	241.78	-	240.01	239.31
8	Waddington	241.88	241.87	241.37	-	239.62	-
9	Morrisburg	241.61	241.55	240.94	240.87	239.19	238.05
10	Long Sault Dam	240.82	240.83	239.80	239.95	238.10	236.95

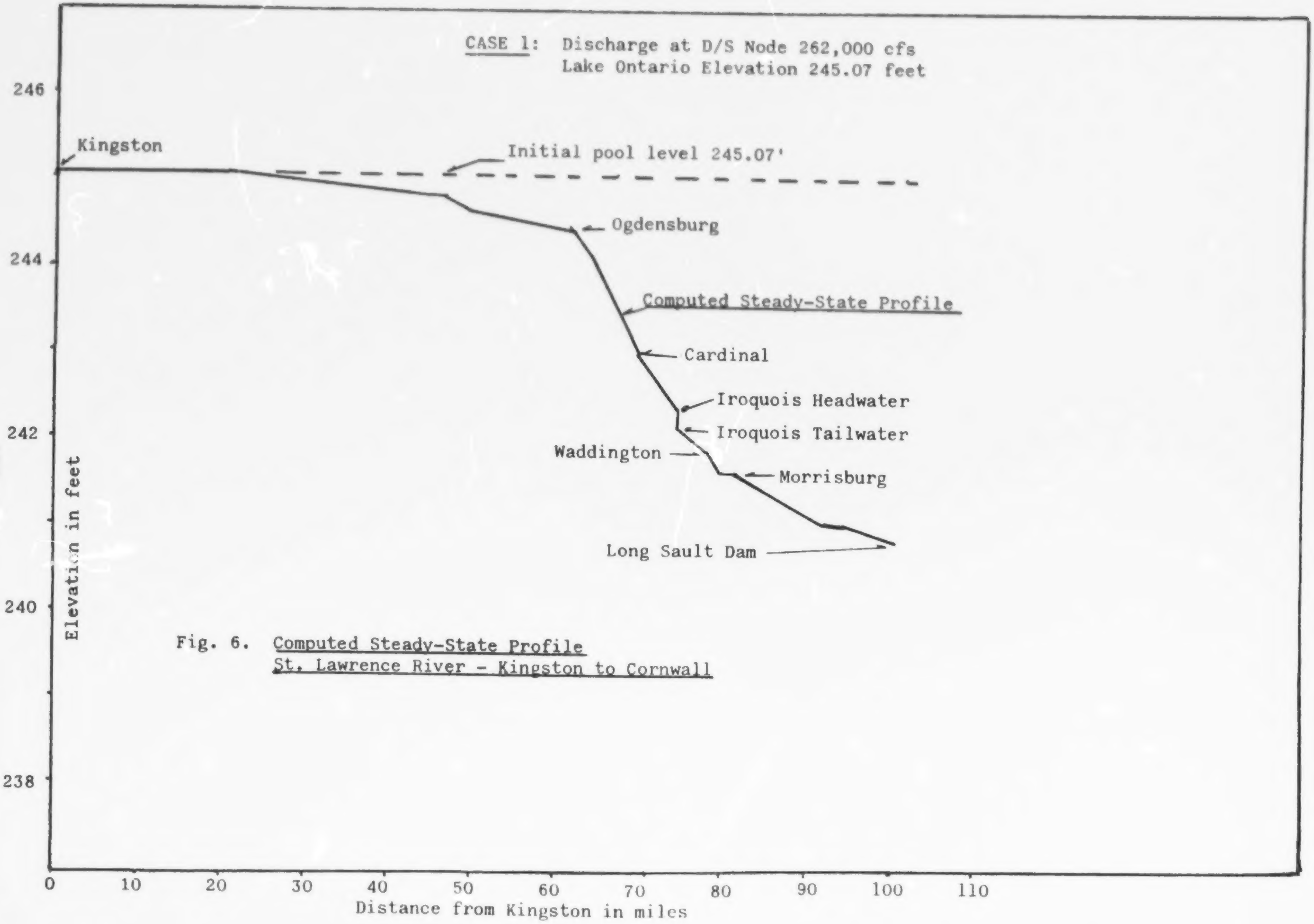


Fig. 6. Computed Steady-State Profile
St. Lawrence River - Kingston to Cornwall

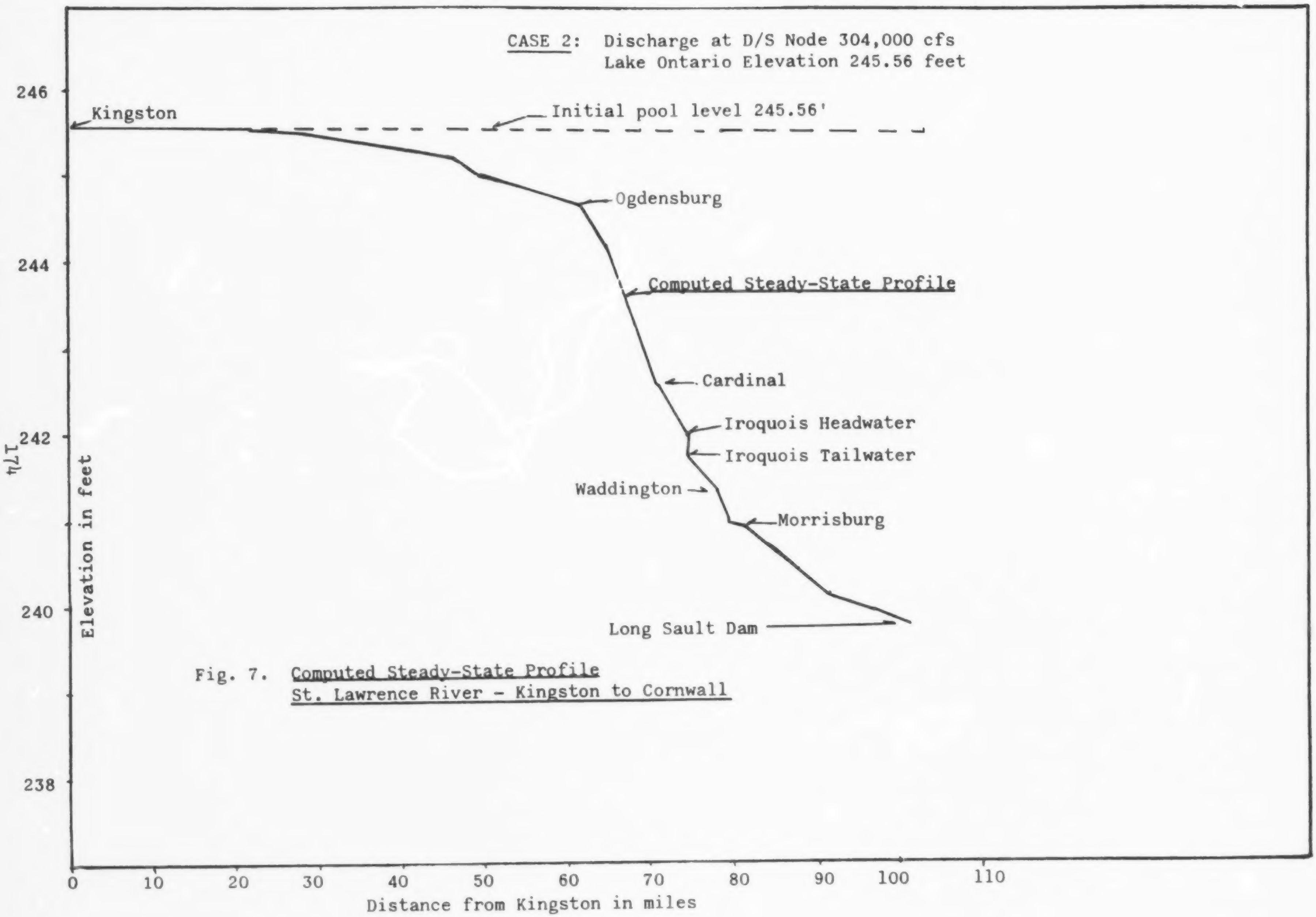


Fig. 7. Computed Steady-State Profile
St. Lawrence River - Kingston to Cornwall

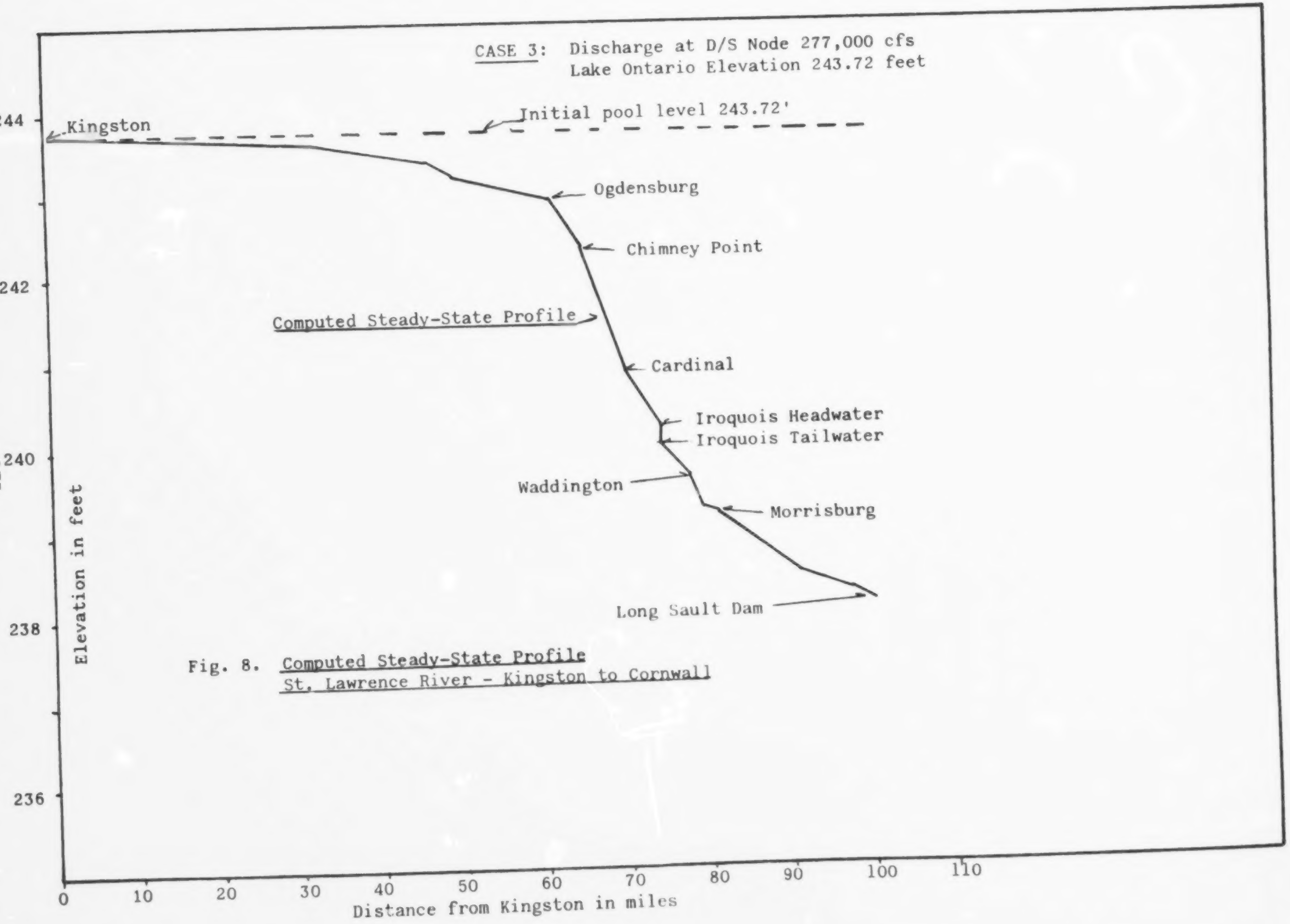


Fig. 8. Computed Steady-State Profile
St. Lawrence River - Kingston to Cornwall

The nonlinear variation of the parameters was not investigated.

A note on Case 3 about the differences in the computed and observed stages is as follows. The total fall of the water surface from Chimney Point to Long Sault dam was computed as 3.13 feet for 266,000 cfs, 4.33 feet for 304,000 cfs and 4.23 feet for 277,000 cfs. The observed falls for the first two cases is almost the same. For Case 3, it was observed as 5.34 feet, which is a greater fall than for 304,000 cfs. The only observation that can be made is that different datum was used in reporting the observed stages.

Summary and Conclusions

A finite element model, based on the solution of the Saint-Venant equations is developed, capable of simulating flow conditions in open-channel networks. The computerized model utilizes a Predictor-Correction solution scheme and is found to be convergent and stable. The solutions are based on linear approximations and the Galerkin method is found suitable in most of the test cases. A variety of problems were tested on the model and the following conclusions are drawn:

1) The model is capable of handling open-channel network problems and simulating the unsteady flow hydraulics therein.

2) Almost any configuration of network can be handled as long as a set of initial and boundary conditions are stipulated.

3) The results obtained by utilizing linear approximations were found to be acceptable.

4) A weighting factor of 1.0 was found to give rapid convergence for any problems requiring a steady-state solution. A weighting factor of 0.55 gave good results for all unsteady flow problems.

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Krishnappan (20)

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MATHEMATICAL MODELLING OF UNSTEADY FLOWS IN MOBILE BOUNDARY CHANNELS

by

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Hydraulics Research Division
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March 1979

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ABSTRACT

In this paper, a numerical model is described which solves the unsteady flow equations for a mobile boundary channel. This model is capable of predicting the flood-wave characteristics as well as the environmental impacts of man's activities in river basins. The model was verified under laboratory conditions and a reasonalby good agreement between the model predictions and laboratory measurements was observed.

MATHEMATICAL MODELLING OF UNSTEADY FLOWS IN MOBILE BOUNDARY CHANNELS

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1.0 GENERAL

Open-Channel Hydraulic Modelling in the context of flood damage reduction activities has to deal with the problem of predicting the time and magnitude of flood peaks occurring at various locations along a river as a flood wave propogates downstream. Such predictions can serve two very valuable purposes: one is to provide the capability of flood-wave forecasting and the other is to provide information for the design of structures that would contain the floods to within a certain predetermined portion of the flood plains.

Prediction of the flood-wave characteristics in rivers can be made by solving the unsteady flow equations developed by De Saint-Venant (1) as early as in 1871. There are numerous methods of varying complexity to solve these equations. Almost all of these methods suffer from one common drawback, namely, inadequate representation of friction slope which appears in one of these equations. In the majority of the existing methods, the friction slope was evaluated using either Chezy's coefficient C or Manning's coefficient n. Both of these empirical relationships were developed some 200 years ago. These coefficients, while representing the frictional characteristics of rigid boundary channels with reasonable accuracy, fail to describe the friction slope in natural channels which transport sediment. Indeed, as can be seen in Figure 1, where Manning's n is plotted against the Reynolds number ($Re=uy/\nu$, u is the average velocity, y is the flow depth and ν is the kinematic viscosity) by R. A. Baltzer and C. Lai (2) for a natural stream near Rio Vista, California, the points exhibit a large scatter especially at low Reynolds number range. For flows in this range of Reynolds number, the estimation of Manning's n becomes very unreliable.

The relative error in estimating Manning's n or Chezy's C reflects in the relative errors in the predicted flow rate and/or flow velocity, which are, unfortunately, of the same magnitude. Figure 2, produced by J. A. Liggett and W. H. Graf (3), exemplifies this fact. This plot was produced for the Detroit River. It can be seen from this figure that a 10% error in the estimation of

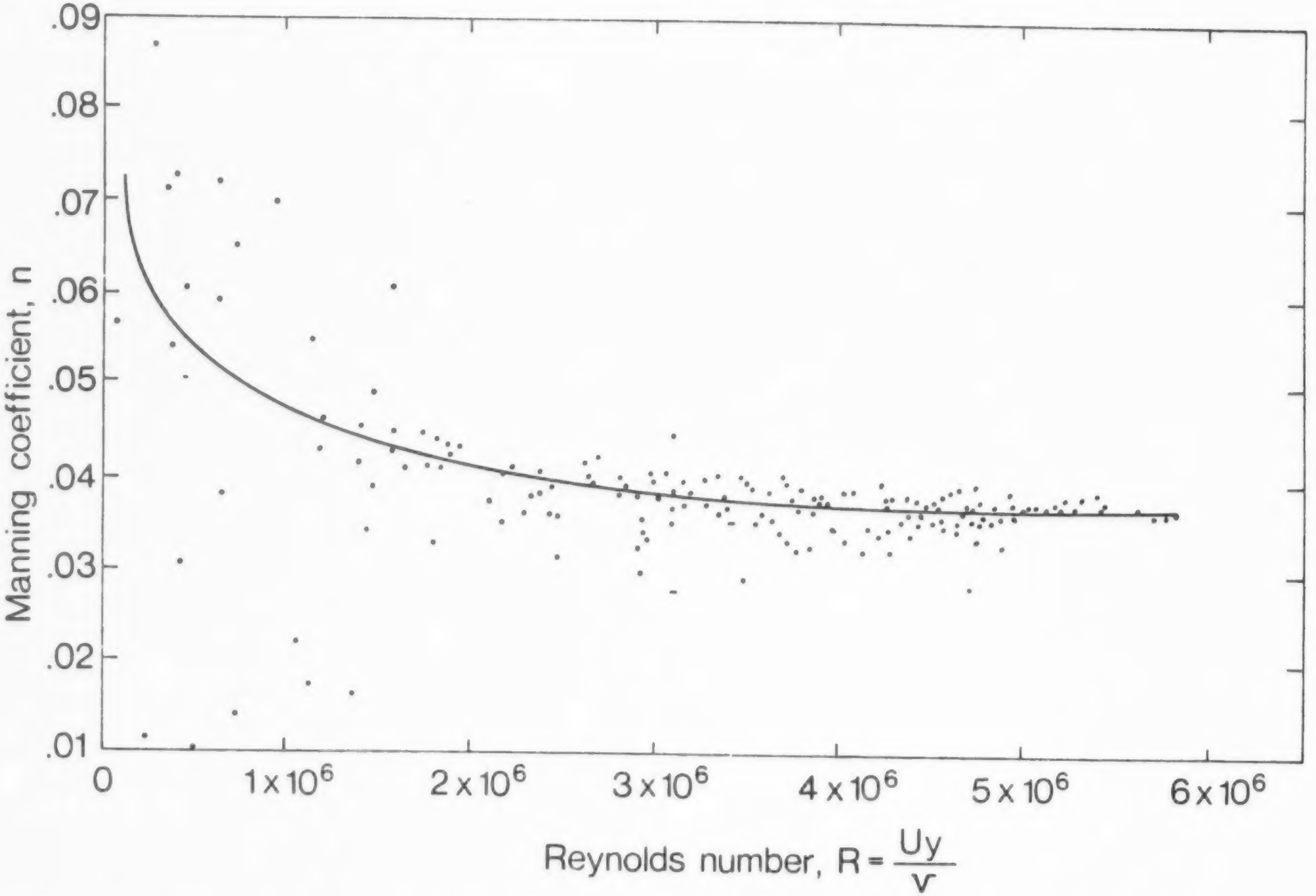

Figure 1 MANNING n vs. REYNOLDS NUMBER FOR THREE MILE SLOUGH NEAR RIO VISTA, CALIFORNIA. (after Baltzer and Lai (2))

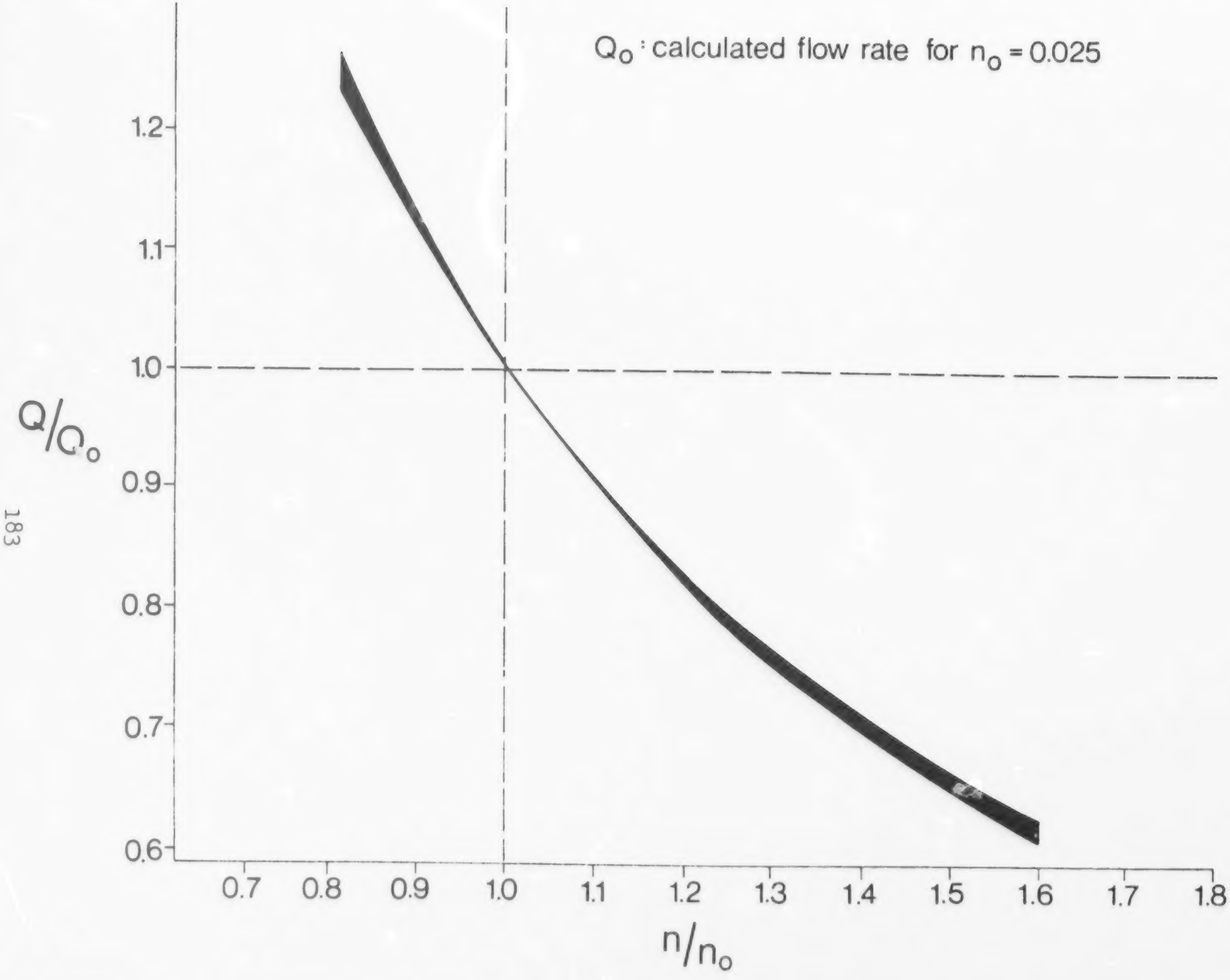

Figure 2 Variation of flow rate with n for the Detroit River (after Liggett and Graf (3))

Manning's n produces a 10% or more error in the predicted flow rate. It also suggests that, in terms of percentage error, it is better to overestimate the value of n than to underestimate it.

In the case of streams transporting sediment, the friction coefficient is governed, to a large extent, by the geometric properties of the sand waves which form at the river bottom. The sand-wave geometry changes depending on the strength of the flow and hence the friction coefficient cannot be represented by a single constant; but it varies as a function of characteristic properties determining the two-phase phenomenon. More details on this topic can be found in M. S. Yalin (4).

In this paper, a numerical method is outlined to solve the unsteady flow equations in mobile boundary channels, with the friction coefficient being specified by a method which takes into account the presence of sand waves at the river bottom. In addition, the present model also considers the sediment mass balance within a control volume, thereby providing the mean bed-level changes in addition to the water level and flow rate variations as a function of time and distance along the river. This model can, therefore, be used for predicting the environmental impacts of the various activities of man in a river basin which could alter the sediment supply to the stream and consequently its regime. In fact, the original intention for the development of the present model was to address the above-mentioned problem. That is why the example chosen to verify the model which will be described in this paper later on, differs from the usual examples on flood wave prediction problems.

2.0 GOVERNING EQUATIONS

A schematic representation of the longitudinal profile and a typical cross section of a river for which the model is applicable is shown in Figure 3. The coordinate system and some of the symbols used in the model are also shown in Figure 3. The governing equations are as follows:

$$\frac{\partial Q_s}{\partial x} + P \left(\frac{\partial z}{\partial t} \right) p + B.Cav \left(\frac{\partial y}{\partial t} \right) + A \left(\frac{\partial Cav}{\partial t} \right) - q_s = 0 \qquad (1)$$

$$\frac{\partial Q}{\partial x} + B \left(\frac{\partial y}{\partial t} \right) + P \left(\frac{\partial z}{\partial t} \right) - q_\ell = 0 \qquad (2)$$

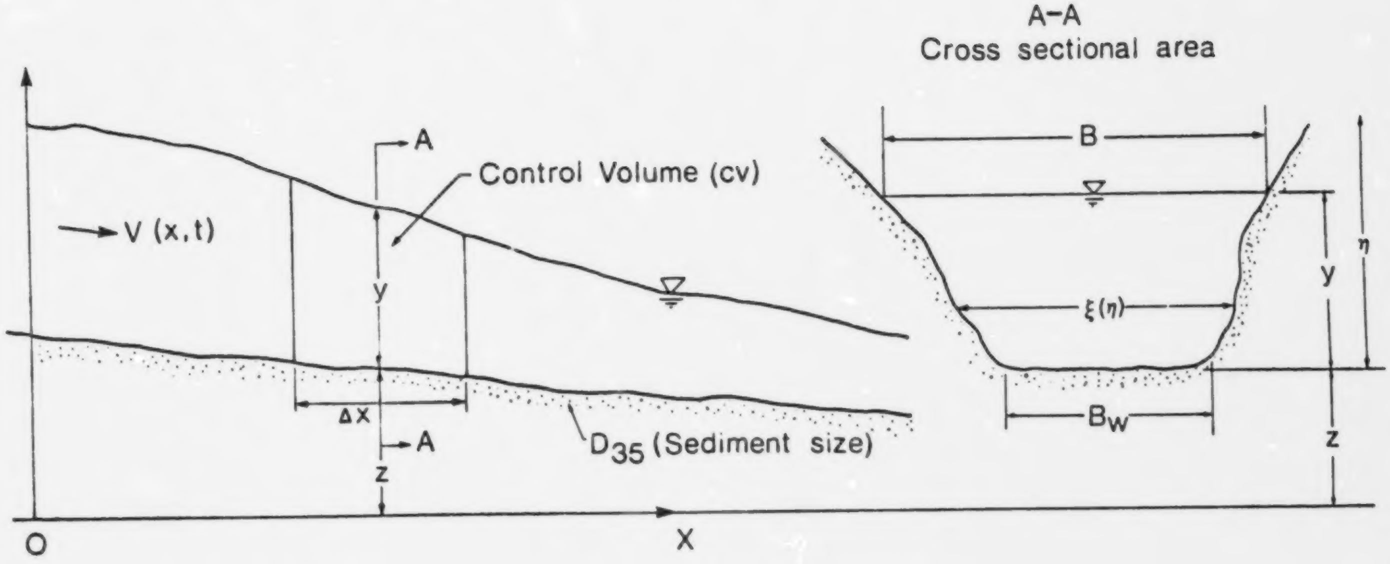
A–A
Cross sectional area
A
Control Volume (cv)
V (x,t)
y
Δx
A
z
D_{35} (Sediment size)
O
X
B
ξ(η)
B_W
y
η
z

Figure 3 SCHEMATICAL REPRESENTATION OF THE LONGITUDINAL PROFILE AND A FLOW CROSS-SECTION IN A RIVER.

$$\frac{\partial Q}{\partial t} + 2\frac{Q}{A}\frac{\partial Q}{\partial x} + g\,A\,\frac{\partial y}{\partial x} - B\frac{Q^2}{A^2}\;\frac{\partial y}{\partial x} = g\,A\,(S_o - S_f) + \frac{Q^2}{A^2}\,A_x^{\eta} \qquad (3)$$

where	x	is the co-ordinate axis along the length of the channel
	z	is the vertical distance between a fixed datum and the mean bed level within a control volume
	y	is the flow depth
	η	is the vertical co-ordinate measured from the mean bed level within a control volume
	t	is time
	g	is acceleration due to gravity
	Q_s	is the volumetric sediment transport rate
	q_s	is the sediment input rate entering the channel laterally
	Q	is the total flow rate
	q_ℓ	is the lateral ℓ inflow rate
	P	is the wetted perimeter
	B	is the top width of the stream
	A	is the flow cross-sectional area
	A_x^{η}	is the rate of change of A w.r.t. x when μ is held constant
	S_o	is the slope of the channel bed within a control volume
	S_f	is the slope of the energy grade line
	p	is the volume of sediment on the bed per unit volume of bed layer
	Cav	is the average volumetric sediment concentration within a control volume.

Equation 1 is the sediment continuity equation which expresses the mass balance of sediment for a control volume. Equation 2 is the continuity equation for the flow and Equation 3 is the momentum equation. The details of derivation of these equations can be found in B. G. Krishnappan and N. Snider (5).

The above set of equations involve six unknowns, namely, the flow rate Q, the flow depth y, the bed elevation z, the sediment transport rate Q_s, the average sediment concentration Cav and the frictional slope S_f. (The lateral inflows, q_ℓ and q_s and the porosity p are assumed to be known). Therefore, in addition to these governing equations, three more independent relations are required to achieve closure of the system of equations. These additional relations are provided by the sediment transport formulae which give Q_s and Cav in terms of flow and sediment characteristics and the equation for friction factor in natural streams which expresses the energy slope S_f in terms of the flow and sediment characteristics. There are a number of sediment transport and friction factor formulae in the literature, but each of them is limited in scope and there is, as yet, no theory that is capable of predicting the above parameters for the whole of the flow regimes. For the present model, the sediment transport formulae of Ackers and White (6) and the friction factor relations of Kishi and Kuroki (7) are adopted, which can be considered the best among the currently available theories. The structure of the present model is such that as new and more complete theories on sediment transport and friction factors become available, they can be easily incorporated into the model.

3.0 NUMERICAL SCHEME

The governing equations can be uncoupled if the term $P(\partial z/\partial t)$ in the flow continuity equation (Equation 2) is considered negligible in comparison to the term $B(\partial y/\partial t)$. Indeed, since the top width B and the wetted perimeter P are nearly equal for wide channels and since the water-level changes are more rapid than the mean bed-level changes, it is possible to drop the term $P\,(\partial z/\partial t)$ from Equation 2 without losing accuracy. By doing so, it is now possible to solve the flow continuity equation and the momentum equation simultaneously for one time step independent of the sediment continuity equation and then to use the sediment continuity equation to correct the solution. Such a technique, which simplifies the solution procedure considerably was adopted for the present model.

The governing equations were solved using an implicit finite difference scheme first developed by Preissmann (8) in 1960. According to this scheme, a variable, say f, and its derivatives are discretized as follows:

$$f\,(x,\,t) = \frac{\theta}{2}\left[f_{i+1}^{j+1} + f_{i}^{j+1}\right] + \frac{1-\theta}{2}\left[f_{i+1}^{j} + f_{i}^{j}\right] \qquad (4a)$$

$$\frac{\partial f}{\partial x} = \frac{\theta}{\Delta x}\left[f_{i+1}^{j+1} - f_{i}^{j+1}\right] + \frac{(1-\theta)}{\Delta x}\left[f_{i+1}^{j} - f_{i}^{j}\right] \qquad (4b)$$

$$\frac{\partial f}{\partial t} = \frac{1}{2\Delta t}\left[f_{i+1}^{j+1} - f_{i+1}^{j}\right] + \frac{1}{2\Delta t}\left[f_{i}^{j+1} - f_{j}^{i}\right] \qquad (4c)$$

where θ is a weighting coefficient that can take values between 0 and 1. When $\theta=0$, the scheme is fully explicit and if $\theta=1$ the scheme is fully implicit. J. A. Cunge (9) had studied this scheme for numerical stability and accuracy and had shown that the scheme is unconditionally stable for values between 0.5 and 1.0 and the accuracy is first order w.r.t. Δx for arbitrary values of θ and second order of $\theta=0.05$. Cunge also indicated that for $\theta=0.50$ "parasitic" oscillations were found in the solution that resembled the phenomenon of numerical instability and had suggested a practical range for the values of θ from 0.60 to 1.0. The present model uses a value of 0.60 for θ.

When the approximations expressed by Equation 4 are substituted in the governing equations, a set of algebraic equations result. These equations are then linearized and solved using a method known as "Double Sweep Method". For complete details, the interested readers can refer to the report by B.G. Krishnappan and N. Snider (5).

4.0 MODEL VERIFICATION FOR LABORATORY CONDITIONS

The model was tested in a sediment transport flume at the Hydraulics Laboratory of the National Water Research Institute. The flume is 22.8 m long and 2 m wide. The slope of the flume bed can be varied anywhere between +0.6 % to 1.0 %. The flume is equipped with a sediment feeder at the upstream and sediment traps downstream. A carriage for the instruments, driven by an electric motor, traverses along the length of the flume at three different speeds of 5, 10 and 30 cm/s. The flume bottom was covered with a reasonably uniform sand of size 1.20 mm. The water supply for the flume is from a constant head tank. Water enters the flume through an inlet pipe, a diffuser and a headbox fitted with baffles and flow straighteners. The flow rate is controlled by a butterfly valve fitted to the inlet pipe and is measured using an "annubar flow meter".

The model was tested for different sedimentation processes such as aggradation and degradation by changing sediment supply and keeping the water supply rate constant and also by changing the water supply rate and keeping the sediment supply constant. Due to lack of space, only one of the tests will be described in this paper. In this test, the water supply rate was varied as shown in Figure 4. The sediment was supplied at a constant rate of 0.06 kg/hr. The water supply rate was changed in steps as can be seen from Figure 4. The water-level

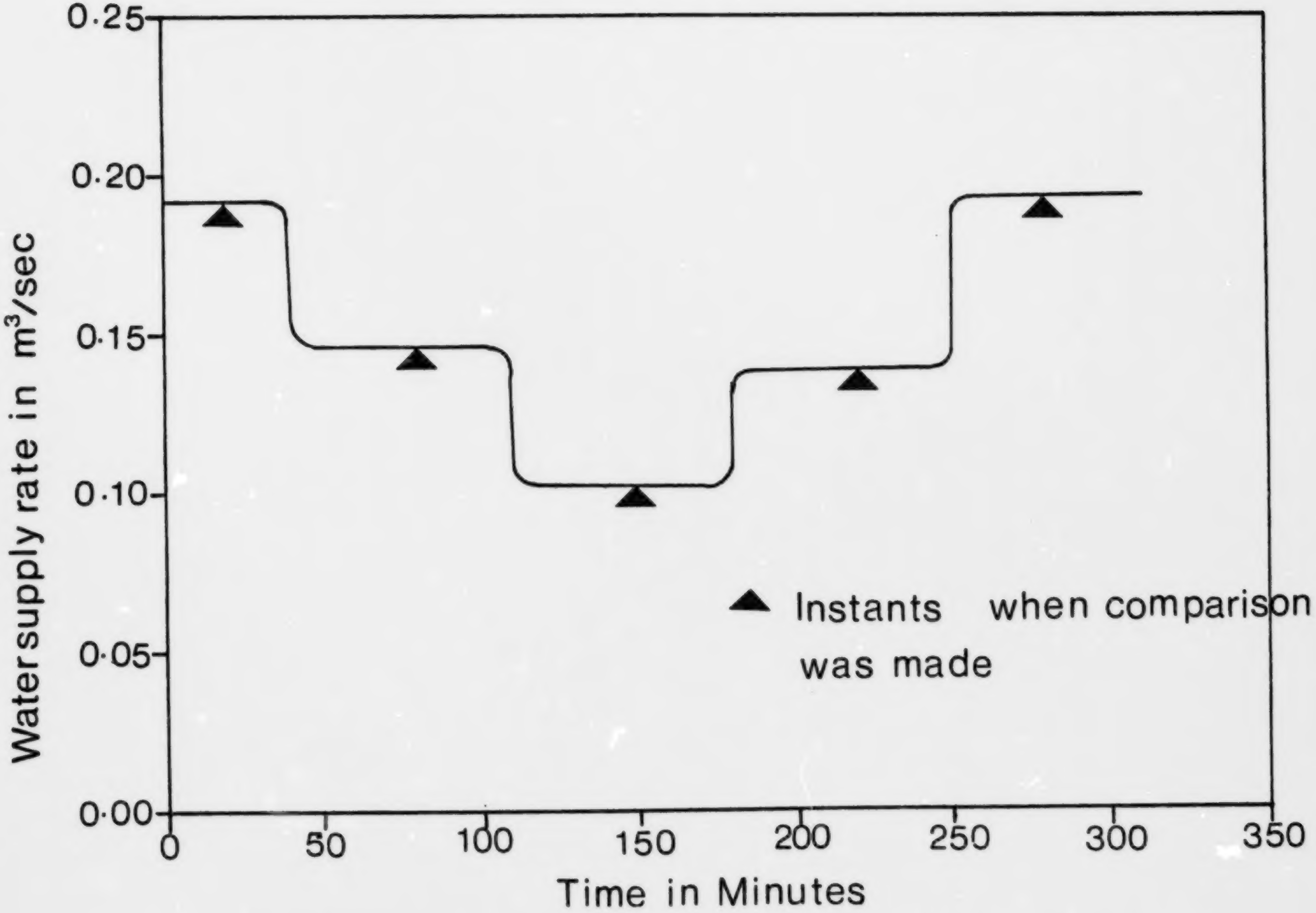

Fig.4 Variation of water supply rate at flume entrance as a function of time

and bed-level changes were recorded by traversing a "water-level indicator" and a "bed-level indicator" mounted on the instrument carriage. The water-level indicator consists of a pointe gauge, the tip of which follows the water level by maintaining contact with the water surface all the time. The bed-level indicator works on the principle of optics and the tip of the indicator maintains a constant gap between the bed surface and the tip and hence traces the bed-level changes. The signals from the bed- and water-level indicators were plotted on a XYY' recorder. The water-lever indicator supplies the water level to the Y channel. The bed-level indicator is connected to the Y' channel. The distance travelled by the carriage was measured by the x channel.

The water level and bed levels were traced every ten minutes. These traces for various instances indicated in Figure 4 are shown in Figure 5 to 10 as continuous lines. Using the water level and bed levels shown in Figure 5 as initial conditions, the model predictions were carried out and are shown in these figures as discrete points. It should be pointed out here, that the model predictions were made for the mean bed level and not for the movement of the sand waves. It can be seen from these figures that the model predictions of the mean bed level and water levels compare with the measurements reasonably well. No attempt was made to compute the mean bed levels from the measured data, but a visual examination of the figures showed that the measured bed and water levels were distributed around the predicted values.

Even though the model predictions were not made for dune shapes and their movement, their effects were taken into consideration in computing the friction parameters using the formulae of Kishi and Kuroki as mentioned earlier. The model, however, is capable of predicting the dune movement. In such cases, the space and time steps have to be very small and hence the length of reach that can be modelled is limited by the computer storage and the time of computations.

5.0 ACKNOWLEDGEMENTS

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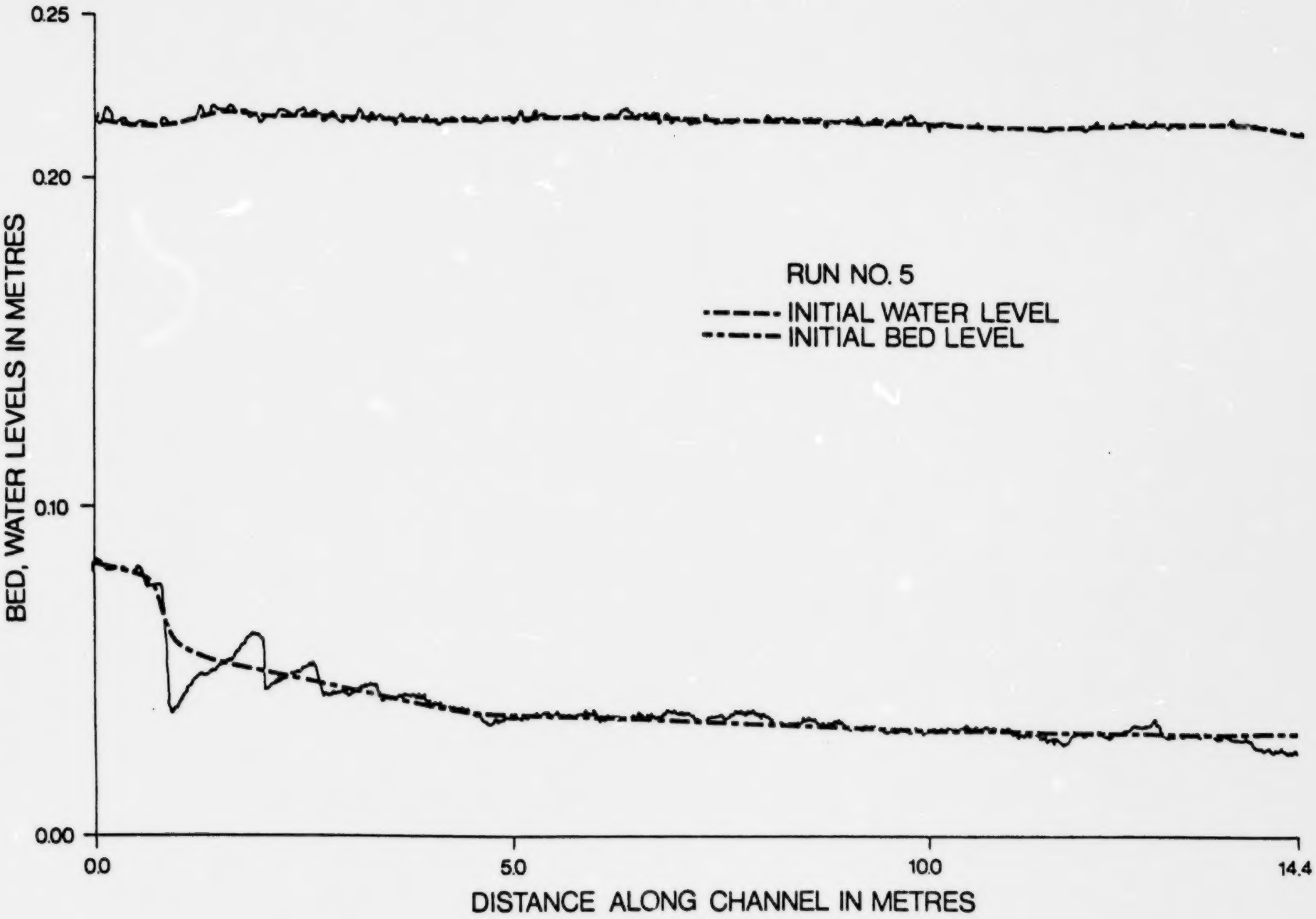

FIGURE 5 INITIAL BED AND WATER LEVELS USED AS INPUT TO MODEL

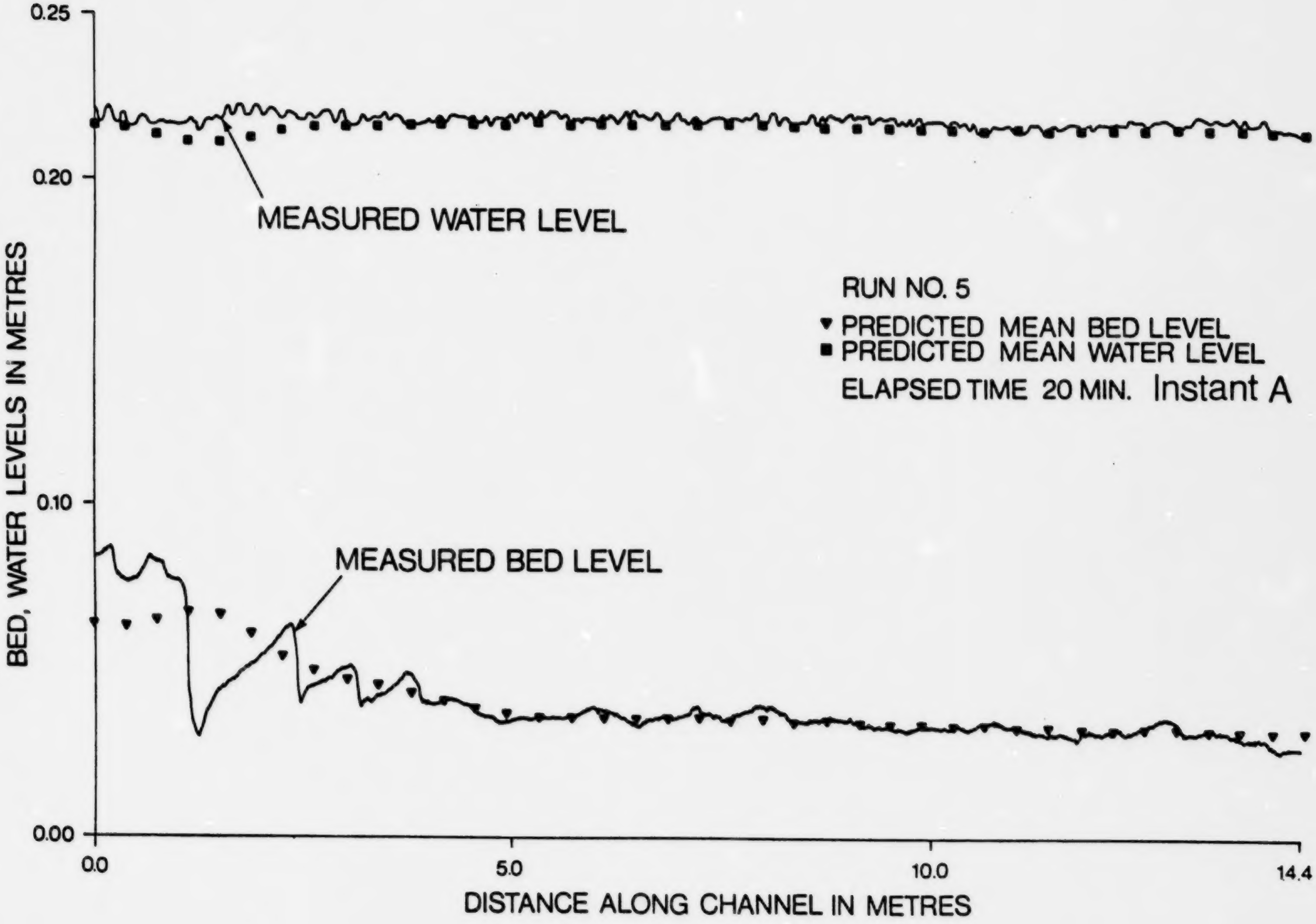

FIGURE 6 COMPARISON OF MODEL PREDICTIONS WITH LABORATORY MEASUREMENTS

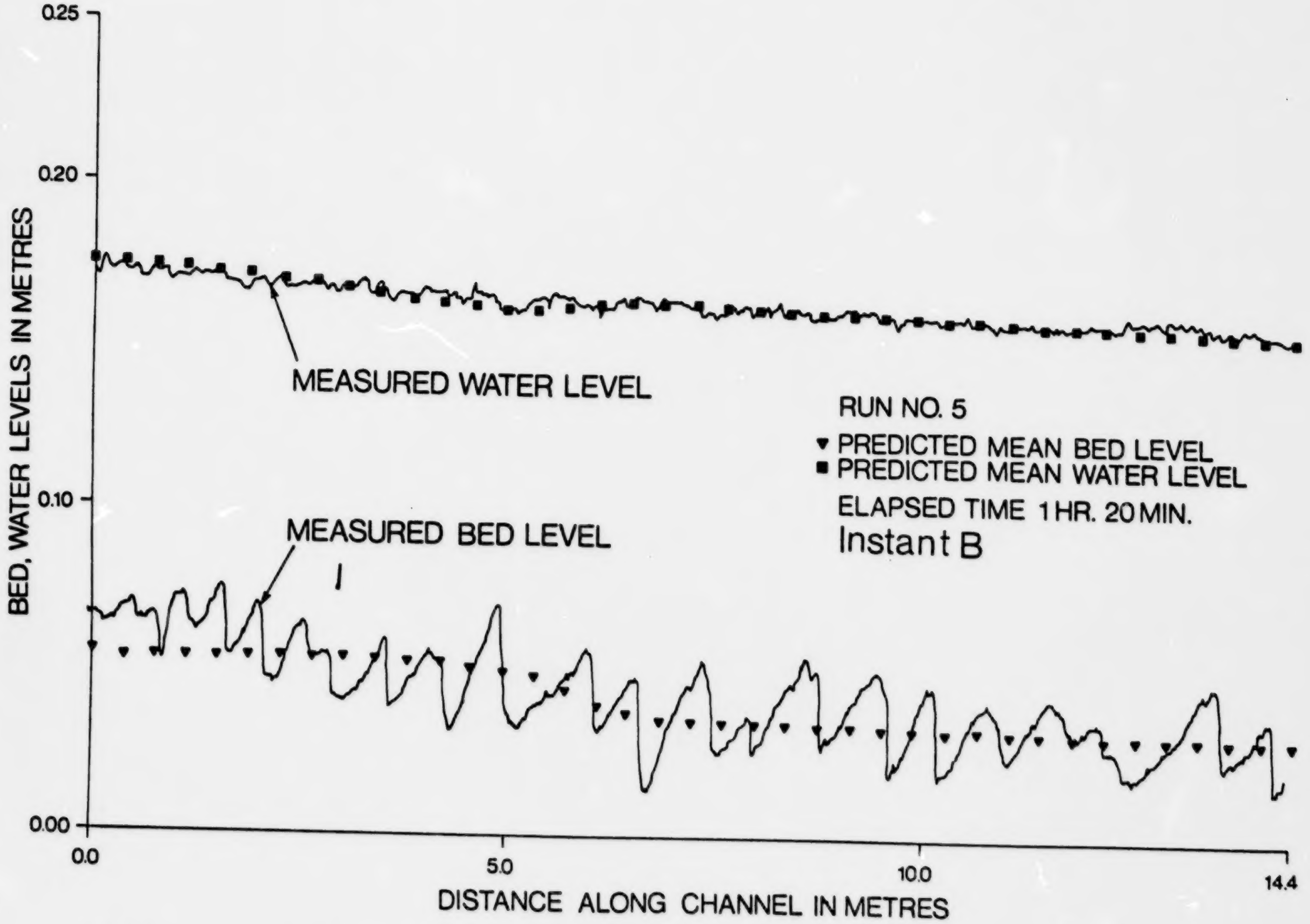

FIGURE 7 COMPARISON OF MODEL PREDICTIONS WITH LABORATORY MEASUREMENTS

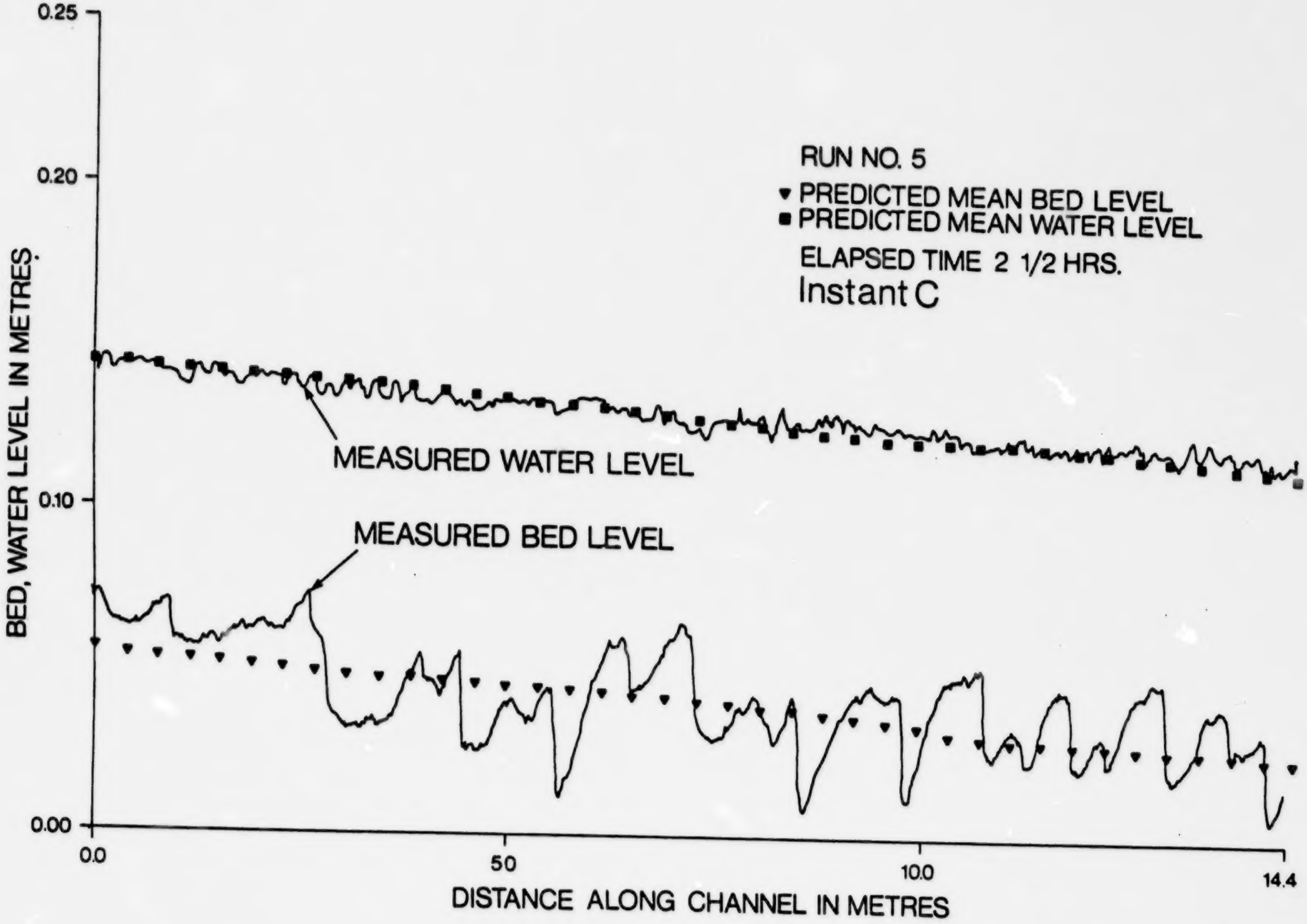

FIGURE 8 COMPARISON OF MODEL PREDICTIONS WITH LABORATORY MEASUREMENTS

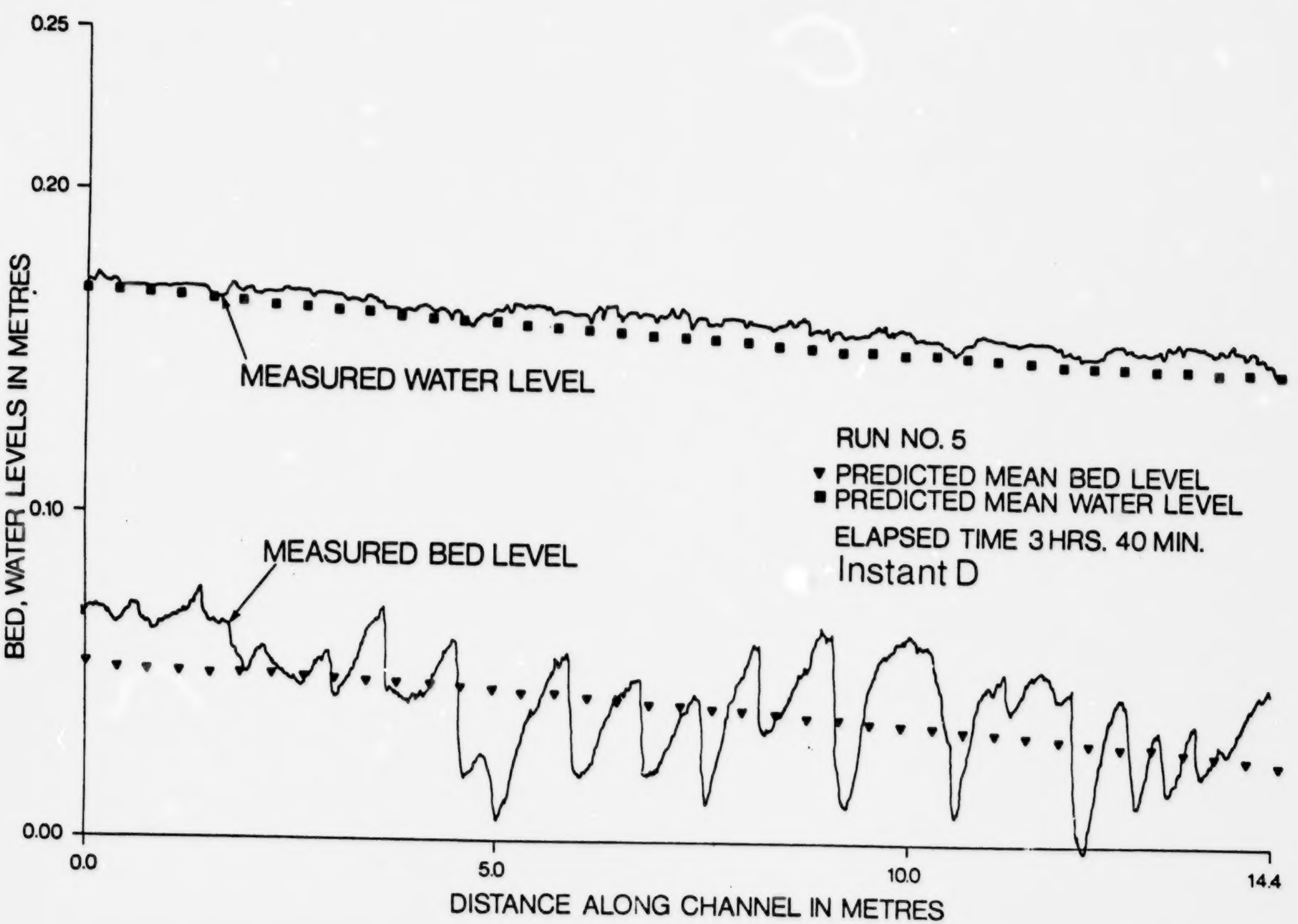

FIGURE 9 COMPARISON OF MODEL PREDICTIONS WITH LABORATORY MEASUREMENTS

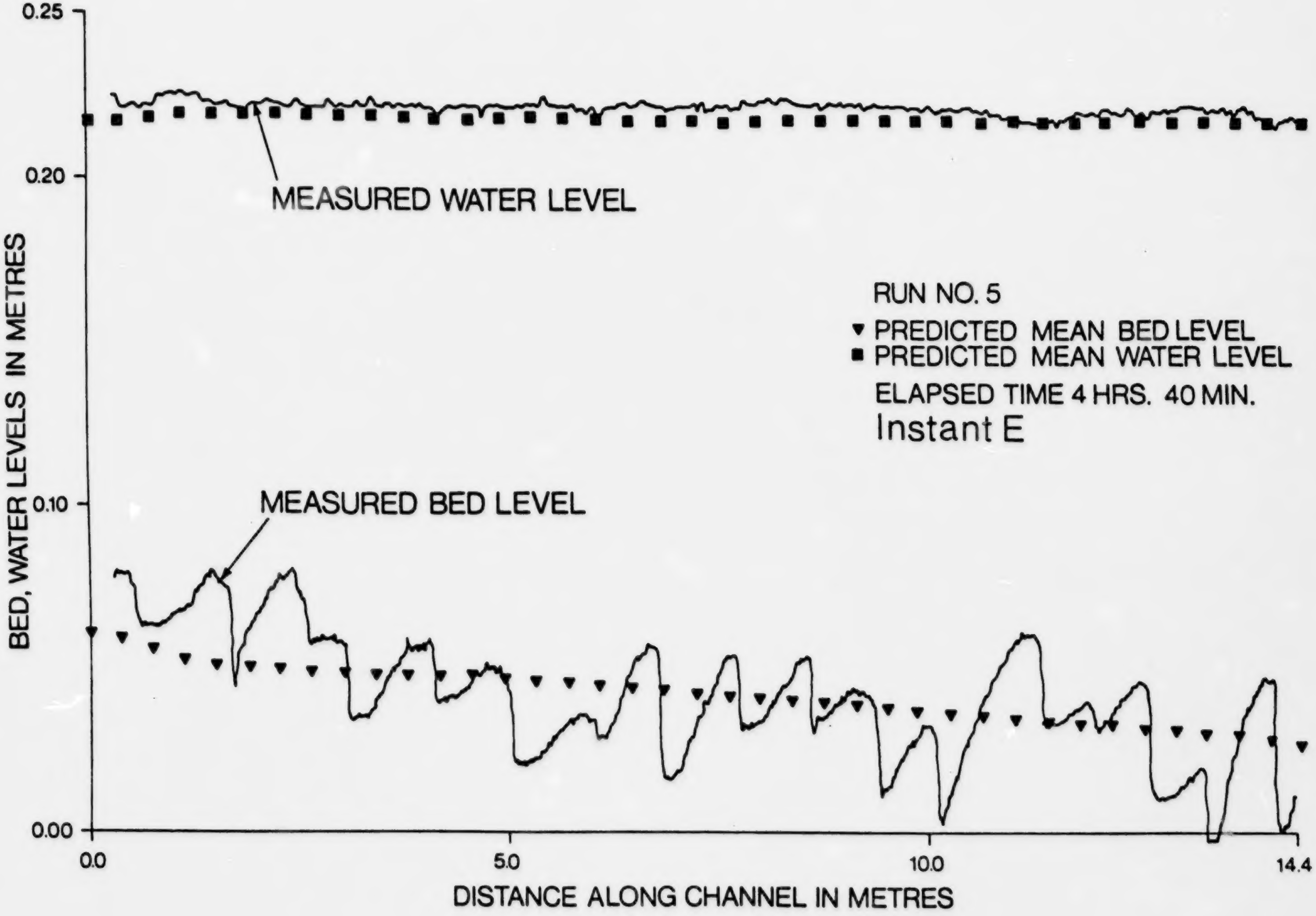

FIGURE 10 COMPARISON OF MODEL PREDICTIONS WITH LABORATORY MEASUREMENTS

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ICE JAMMING CONCERNS
AND
OPERATIONAL SOLUTIONS

Prepared for the
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E. Langley

Water Planning & Management Branch
Inland Waters Directorate
Environment Canada
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ICE JAMMING CONCERNS AND OPERATIONAL SOLUTIONS

The maximum levels reached by rivers in Atlantic Canada occur during the spring, normally at break up time and not when river discharge is a maximum for ice-free flow. The extent of spring flooding therefore depends much on ice jamming. In any year, the formation of ice jams is dependent on the timing and intensity of the warm-up period and on the condition of the ice cover at the time of break up. An early and rapid snowmelt accompanied by rain causes river discharges to increase while the ice cover is still intact. In many river reaches, the additional shear force (on the underside of the ice cover) generated by the increased flow velocities is sufficient to break up the still competent ice cover. The ice migrates downstream and, depending on the configuration of the river channel, may jam at local constrictions such as bridge piers, narrow channel reaches, islands, shoals, sharp bends or other sections of stable ice cover. Once firmly wedged, the ice jam gradually grows in length in the upstream direction and its thickness may increase due to deposition of ice on its underside. The jam impedes the free flow of the river and causes upstream water levels to rise. As the jam grows, water forces acting on it increase until sufficient to cause the ice jam to shove and move downstream once more.

Flood levels during a spring thaw period are thus not always merely a function of river discharge and channel characteristics, but also of the constrictions caused by ice jams.

Ice jams have occurred at frequent intervals on all major rivers and streams in New Brunswick. The Saint John River experiences ice jams practically every year in the upper reaches at Nine Mile Bridge. Farther downstream, approximately six jams have occurred in the last century in the Woodstock area. It is interesting to note that since the construction of the Mactaquac reservoir, no ice jams have occurred downstream from this point, whereas upstream, at Woodstock for example, ice jams still occur.

Two types of ice jams occur in Nova Scotia where water levels in the flood plain are influenced by the sea. The first type occur in rivers due to normal ice formation in channels and thus cause flood conditions from backwater. The second are ice jams caused by sea ice being driven by winds and tides into the estuary areas blocking the passage of river ice and discharge. To compound the problem, the province may experience several freeze and thaw periods during the winter, as it did this year, resulting in more than one break up and increased chances for ice jam floods.

Ice jam floods in the Atlantic Provinces are often more severe than the open water floods. For example, the 1976 ice jam flood at Perth-Andover on the Saint John River was more extensive than the 1973 flood, the flows for which had a recurrence interval considerably higher than 100 years.

The problem of considering ice jam floods for flood risk mapping is that ice jams occur at various places in the river channels and estuaries and at times are unpredictable. The extent of flooding will vary considerably depending upon the timing and location of ice jams. However, because of their significance, ice jam floods should be considered for flood risk mapping. The problem arises in including ice jam flood levels in the frequency analysis required to determine the flood flows for different return periods and in considering the ice jam situation in the hydraulic work. Including ice jam floods in these calculations is difficult and sometimes impossible due to the lack of historical data. There are no set procedures set out in the "Hydrologic and Hydraulic Procedures for Flood Plain Delineation", by Environment Canada. Each area is being considered separately taking into consideration the significance of the ice jam flood, ie. was it greater than the 1:100 year open water event, factors other than ice jams to be considered in the hydrotechnical work, the available data and a reasonable time and cost expenditure to solve the problem.

SOME ICE JAM PROBLEM AREAS

A couple of examples of ice jam problem areas and how we are treating the situation are discussed herein.

Perth-Andover

The village of Perth-Andover is situated on the banks of the Saint John River approximately 1.5 miles downstream of the confluence of the Saint John and Tobique Rivers, and is bisected by the headpond of the Beechwood power dam. The village suffered its worst flooding in living memory in the spring of 1976 as a result of severe ice jamming. Damage to property was considered to be extensive. Water levels peaked about 10 feet above normal levels.

Under the Flood Risk Mapping Agreement signed with New Brunswick in March, 1976, a study was undertaken to determine the 1:20 and 1:100 year open water flood lines for flood plain delineation. When plotted on detailed maps, the 1:100 year flood affected a very small area. In fact, in many places, the flood line was on the river bank. The significant floods were the ice jam events. Therefore, it became important to consider ice jam floods. Considering the difficulty of determining the frequency of ice jamming in various areas of the flood plain, the technical committee decided to map the 1976 ice jam flood.

There is extensive (or suitably accurate) documentation of the 1976 event. Data, including oblique air photos, water levels, and newspaper articles, were obtained from various sources but primarily from New Brunswick Electric Power Commission and supplemented by field surveys for the delineation of the flood line.

According to the "Hydrologic and Hydraulic Procedures for Flood Plain Delineation", the effects of ice jams on a flood of a given magnitude should have been considered but this was not practical due to the lack of available data.

In view of the experience at Perth-Andover in flood risk mapping, an ice jam observation program was established for selected areas of New Brunswick. The New Brunswick Electric Power Commission, Inland Waters Directorate, University of New Brunswick, Environment New Brunswick and the New Brunswick Department of Transportation are cooperating to obtain reliable data on ice thickness (snow ice, black ice, and white ice), depth of water, distance from bottom of the ice to the top of the water and depth of water under the ice, in an attempt to predict ice break up. In addition, for all large flooding events, data is being collected to describe the areal extent and height of flooding. This program includes photography to provide a visual record of the flood, elevation observations by reading crest gauges and marking water surface elevations on structures and staking at intervals along the contour of the highest water surface elevation of the flood event such that detailed surveying could be done later.

Truro

Under the Flood Risk Mapping Agreement signed with Nova Scotia, June, 1978, Truro has been given number one priority. Presently, Terms of Reference for a hydrotechnical study are being drafted. One of the problems encountered is the treatment of flooding due to ice jams.

Ice jams in the Salmon and North Rivers contribute significantly to the flooding problem in the Truro area. Ice in tidal rivers such as these, have two origins. Here, sea ice is formed on the shoals of Cobequid Bay and this ice is carried into the estuary, with the incoming and rising tide, and by the prevailing onshore wind, where it is stranded on the banks. The tidal channel becomes smaller and smaller and is finally choked off completely during a very cold spell. In addition, ice is formed in the fresh water section of the river and can, during break up, move downstream and accumulate at a constriction either in the fresh water portion of the river or at the sea ice jam. The ice jams are notoriously unpredictable both in time and place of occurrence. When an ice jam occurs, regardless of type, the water keeps on rising during a heavy runoff until it finds a sufficiently large alternative outlet, either around the jam or over it or both, and/or it goes into storage. If an ice jam forms in front of a bridge or culvert, the water will be diverted along the road until it finds a spot where it can return to its normal river bed. This has recently been the case with McLure Brook, a Salmon River tributary, where the culvert under the highway ceased functioning properly because of an ice accumulation in front of the culvert.

Physically there are other factors which contribute to the ice jam flooding problem in the Truro area. The hydraulic gradient of the Salmon River, which is 40 to 80 feet per mile in the fresh water sections, diminishes drastically in the flood plain. During incoming tide, this gradient is reversed. The velocity of the water flow decreases in this section and the surface area of the river becomes larger. This provides more opportunity for ice formation and accumulation. Also, there is a problem with tidal silting reducing the channel cross-section during periods of low and mean velocities. During heavy discharge, eroded upland material is carried downstream sometimes forming gravel bars on which the freshwater ice jams.

As a first step in the assessment of ice jamming problems as related to flooding events in this area, an annotated bibliography was prepared in 1977 by Water Planning and Management Branch. The literature search included: 1) the causative factors leading to ice jamming, 2) the physical properties of ice and ice cover, 3) the occurrence of ice covers and related events, 4) observation of ice, and 5) analysis and application of methods.

In 1978, the National Water Research Institute studied water levels obtained during the floods with and without ice jams and concluded that there was no significant difference. However, local people and professionals familiar with the area firmly believe that ice jam flood events are more severe than floods without ice jams.

This past winter, a data collection program was initiated. The purpose of this was to gather information which would be useful for upcoming mapping, such as a definite locational pattern of ice jams. Data collected included pictures; location, size, movement and duration of ice jams; ice conditions; ice thickness; gravel build up and bank erosion. We realize this data is limited in scope but we hope it will result in an increased understanding of ice jam flooding in the Truro area. Both ice jams that occurred this past winter were in approximately the same location; however, it is expected that this survey should be carried out over a few years before meaningful conclusions can be drawn from the data.

As yet, the Flood Risk Mapping Technical Committee, formed under the Flood Risk Mapping Agreement has not decided on the best means of handling the ice jam flooding problems for Truro. There is the possibility of (1) mapping a large historical ice jam flood event if it is greater than the 1:100 year flood, (2) carrying out detailed studies subsequent to a data collection program, (3) carrying out studies with the limited available data. Ideally, a detailed study of the mechanisms of ice jamming in Truro and an analysis of the frequency of ice jamming in the various areas, should be carried out. Included in the analysis could be the magnitude of flooding from ice jams in particular areas of the flood plain and an attempt to identify areas where the flooding is more prominent. However, time and cost considerations plus the technical capability of dealing with ice jams along with the other problems, i.e., the changing cross-sections, in the Truro area may necessitate using a simple approach such as mapping a historical event. In fact, considering the quality of the data and the accuracy anticipated in any ice jam modelling procedure, a detailed study may not be warranted. Mapping of a large historical ice jam would be the simplest solution provided it is greater than the 1:100 year flood and there is suitable documentation.

TECHNICAL PROBLEMS

Perth-Andover and Truro are just two of the areas identified for flood risk mapping in Atlantic Canada. Many of the areas identified for Flood Risk Mapping in New Brunswick and most of the areas identified in Nova Scotia have ice jam flooding, some due to both sea and river ice. Although desireable, it is impractical in many areas to carry out detailed studies.

Lack of historical ice jam data is a problem in all locations of concern. Useful data includes: location of past jams; probably cause of jams; jam duration, washouts and reformation; extent of flooding; progression of flooding after a jam; river discharge and water level variations upstream and downstream after jamming; the effect of tides on jamming; meteorological data; ice thickness and strength at breakup; ice cover conditions before break up and river cross-sections. Miscellaneous information may be available from sources such as old newspaper articles, Emergency Measures Organization files and staff and private individuals but no one is collecting this type of data on a regular basis with the exception of measurements of discharge and levels by the Water Survey of Canada and meteorological data by the Atmospheric Environment Service. Unfortunately, the data collected is not necessarily in the location of concern. Little is known on the effects upon ice jamming of culverts, drainage ditches, dykes, bridges, the occurrence of sand and gravel bars in the river channel, changes in the stream profile or channel gradient, and restrictions in the stream channels. To be useful a data collection program should extend over a few years. If this data were to be collected, the time allotted for mapping under the agreements would have to be extended.

Little research has been carried out on the effects of ice jams on flood flows of a given magnitude. Techniques to solve this problem need to be developed. Once modelling techniques are developed with respect to ice jam hydraulics, a more accurate assessment of data needs can be made. Some of the questions that arise in including an ice jam in a model include: where to locate the jam; what size of jam to use; should the worst possible situation be used; and how do we attach a frequency to an ice jam event when there is a lack of data.

The modelling program can be avoided by mapping a historic ice jam event larger than the 1:100 year flood. This can only be done if there is enough documentation on flood levels for such an event. This method may not be as acceptable policy-wise but may be the most practical.

SUMMARY

There is a need to develop guidelines for including the effects of ice jams in flood risk mapping work and an even greater need to obtain reliable information on ice jam events. In the meantime, we propose to consider each location separately as to the best method of including ice jamming problems in flood risk mapping.

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HYDRAULIC ASPECTS OF ICE JAMMING

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HYDRAULIC ASPECTS OF ICE JAMMING

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1.0 THE OCCURRENCE AND CLASSIFICATION OF ICE JAMS

As seasons change, most rivers in Canada also undergo the cyclic change of from open-channel flow to ice-covered flow to open-channel flow again. Accompanying the above changes is the possibility of ice jamming.

The occurrence and classification of ice jams may be shown by following the chart in Fig. 1.

It is seen from Fig. 1 that as winter approaches, the water in an open channel is gradually cooled down and ice is formed in the water. According to the condition under which the ice is formed, there are two kinds of ice. The first kind is the static ice which is formed on the surface of quiet or slow-flowing regions and has large columnar crystal structures oriented in the vertical direction. Static ice first forms near the banks where the water is the coldest and is known as border ice. As the border ice gradually grows and extends into the flow; and the flow is not fast enough to break the extending crystal structure, the ice covers from the two banks will eventually meet and an ice bridge across the river is formed. Sometimes, the ice bridge is formed by an ice floe or several ice floes lodged between the two border ice covers.

The second kind of ice is known as dynamic ice and is formed in fast-flowing water where high turbulence inhibits the nucleation of ice crystals until the water reaches a certain degree of supercool. Dynamic ice first appears as minute crystals, known as frazil crystals, and is uniformly distributed throughout the whole turbulent, supercooled layer. As the frazil crystals slowly float to the surface and agglomerate, frazil slush is produced. Frazil ice may also nucleate on or adhere to underwater objects and grow as anchor ice. As the result of further agglomeration, compacting and freezing, given time, frazil slush may consolidate into small ice floes with upturned edges, often in circular form, as a result of grinding between themselves. Such frazil ice floes are known as pancake ice.

In early winter, the drift ice in a river is mostly of dynamic origins, either as frazil slushes or as frazil floes. Anchor ice, when its bond with the river bottom is weakened by solar radiation, will also float to the surface and join the drift ice on the river. Ice floes of static origin are few. They are mostly broken off from border ice or the trailing end of ice covers.

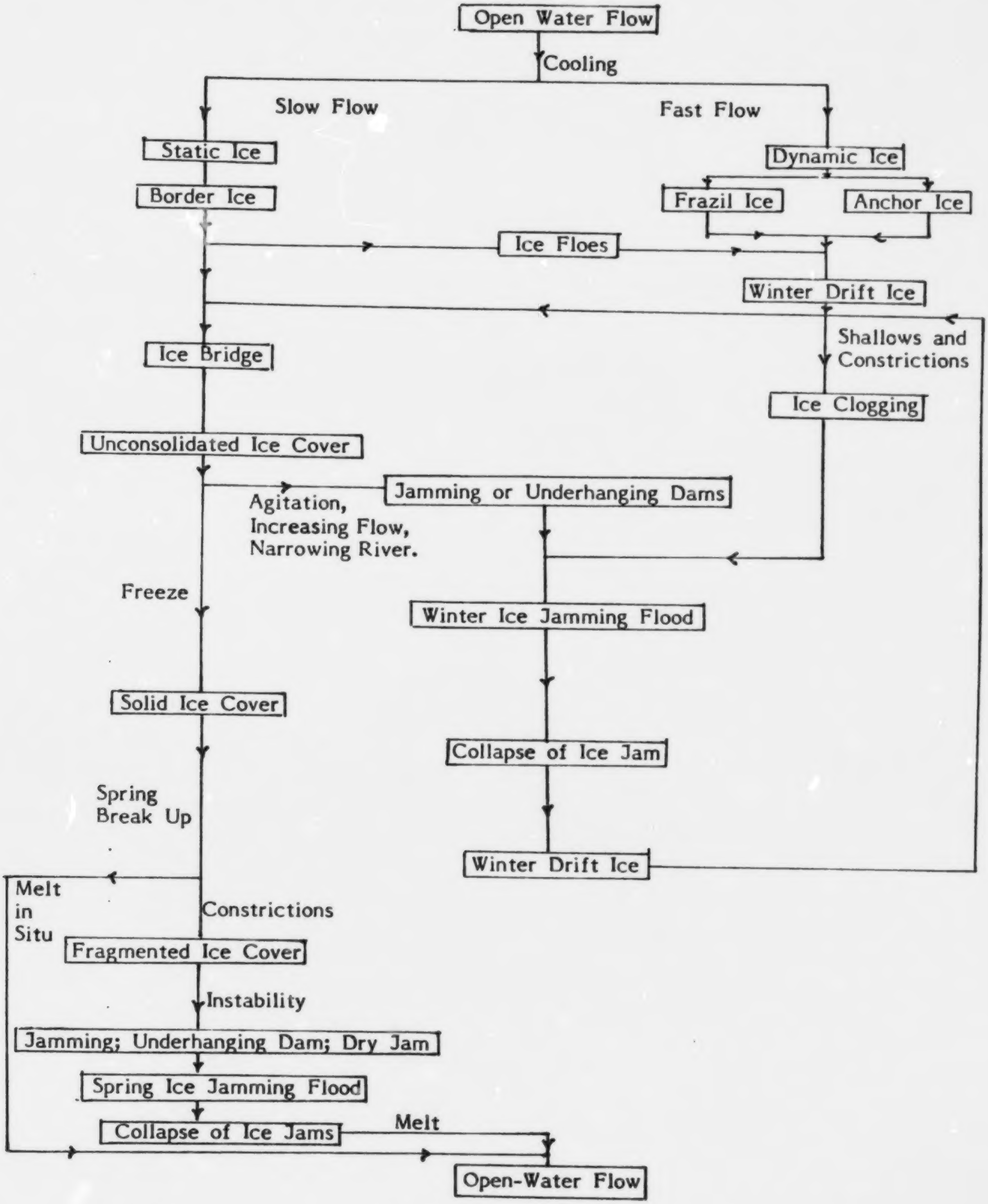

Open Water Flow
Cooling
Slow Flow
Fast Flow
Static Ice
Dynamic Ice
Border Ice
Frazil Ice
Anchor Ice
Ice Floes
Winter Drift Ice
Ice Bridge
Shallows and Constrictions
Ice Clogging
Unconsolidated Ice Cover
Jamming or Underhanging Dams
Agitation, Increasing Flow, Narrowing River.
Freeze
Winter Ice Jamming Flood
Solid Ice Cover
Collapse of Ice Jam
Spring Break Up
Winter Drift Ice
Melt in Situ
Constrictions
Fragmented Ice Cover
Instability
Jamming; Underhanging Dam; Dry Jam
Spring Ice Jamming Flood
Collapse of Ice Jams
Melt
Open-Water Flow

Fig. 1 Life cycle of river ice and ice jamming

When the drift winter ice, consisting mostly of slush ice and small ice floes, reaches an ice bridge, its advancement is stopped. The oncoming ice accumulates in front of the ice bridge and an unconsolidated ice cover is formed and grows. The growth will continue as long as the ice supply is there. The ice cover is consolidated into a solid ice cover when the accumulated ice is compacted and frozen together.

The above situation happens only when the flow is slow. When the flow is very fast, the oncoming ice will be simply swept under the ice cover, either to be deposited onto the ice cover somewhere downstream, or reemerge downstream of the ice cover. Between the above two flow conditions, however, the oncoming ice will be entrained by the current and deposited near the leading edge of the ice cover. An under ice hanging dam thus is formed. The hanging dam grows as more ice is deposited onto it.

The hanging dam greatly increases the local resistance and causes the upstream stage to rise. The flood so produced is known as the winter ice jamming flood.

Although most winter ice jamming floods are caused by underhanging frazil ice dams, sometimes a winter ice jamming flood can be caused simply because too much dynamic ice is produced over a fast-flowing reach and the downstream end of the reach is all clogged up by it.

Since the structural strength of slush ice is low, as the hydrostatic head is built up behind the hanging dam or the jam, the hanging dam or the clogging ice jam will be destroyed without much upstream flooding. However, under certain hydro-meteorological conditions, a frazil ice hanging dam or jam can be solidified and cause major flooding upstream.

Major ice jamming floods occur in the spring. As spring approaches, the accumulated snow begins to melt. During the day, the melting is accelerated and this causes the stage to increase rapidly. During the night, the melting is reduced and this caused the stage to fall. It may be worth noting that ice is a plasto-elastic material, in the way that it deforms readily under a light, but sustained load, although on the other hand it can be quite strong under a load of short duration. Thus, when the stage rises, the ice cover rises with it and detaches itself from the banks. As the stage falls, the ice cover also falls with it and breaks into large pieces under its own weight as it is lodged against the banks. Since the ice floes during spring break up had not been weakened by weathering, they are quite strong besides being of large size.

When the spring ice floes drift down the river and come to certain constrictions, such as sharp bends, shore-fast or bottom-fast ice, bridge piers, islands, narrowings or shallows, the smooth flow of ice is hindered and under certain conditions the ice floes will be stopped by the constriction and block further ice flow. Generally, it is the large floes that first cause the blockage, then the smaller floes add to it. Because spring ice floes are bigger than winter ice floes, everything being equal, ice blockage is more likely to take place in the spring than in the winter.

Once a blockage is formed, further on-coming drifting ice is stopped by it (unless the flow velocity is so high that the on-coming ice floes are swept under it and reemerge downstream) and gradually accumulate in front of it to form an unconsolidated, fragmented ice cover. Such a spring unconsolidated ice cover is different from a winter unconsolidated ice cover in that:

1. It is unlikely to freeze into a solid ice cover.
2. Because of the large amount of ice floes on the river during spring break up, such a fragmented ice cover can be miles in extent.
3. A long, unconsolidated ice cover means that the internal stress in the ice cover is high, so the ice floes suffle, turn and jut, resulting in very rough upper and lower surfaces.
4. As the large spring ice floes ramp on to the upstream edge of a unconsolidated, fragmented ice cover, their large momentum is used to further compact the ice cover, adding to the roughening of the ice cover in front of the restriction.

Such a rough spring fragmented ice cover gives much resistance to the flow and is known as spring ice jam.

If a spring ice jam remains stable, it is known as a static jam. However, if it becomes unstable and starts to crumble, the static jam is changed to a dynamic jam.

The initiation of a dynamic jam can be caused by wind and wave actions, by the increase of the internal stress to a certain critical point, by a sudden increase in discharge, by the propagation of the ice cover to a narrowing reach or by the passage of a ship through the jam.

In the case of instability caused by increasing velocity, either by increasing discharge or by reducing the river width, the oncoming ice floes can no longer be contained by the leading edge of the ice cover, but are entrained by the flow and go down the ice cover, to be deposited near the leading edge and form an under-ice hanging dam. Many times the leading edge itself also becomes unstable and begins to roll in the direction of the flow and contribute to the formation of the hanging dam. Once an under hanging dam is initiated, an unstable state of equilibrium is created because the hanging dam further increases the local flow velocity, which in turn, produces a greater negative pressure differential to pull the ice floes down to add to the dam. The stable state will only be reestablished when the negative pressure is balanced by the buoyancy of the hanging dam. Sometimes the stable state cannot be reached until the dam reaches the bottom and completely blocks the flow. Under such a situation, a dry jam is produced. For a dry jam, the water passes only by filtration.

The initiation of a dynamic jam by mechanical cause is similar to that by increasing velocity, only that the local thickening or the initial hanging dam formation is not necessarily confined to the leading edge.

A spring ice jam gives much resistance to the flow, especially a dry one, therefore rapid upstream stage increase should be expected of a spring ice jam. Since the spring break up ice floes are strong, an enormous hydrostatic head will be needed for its destruction. The most severe ice jamming floods, therefore, occur in the spring.

Existing ice covers downstream often provide the initial blockage needed for the initiation of an ice jam. For north-south rivers, the river ice breaks up from the downstream end. The drifting ice floes, therefore are less likely to encounter a solid ice cover and be stopped by it. Even if this does happen, the hydrostatic head to destruct the jam will be small since the ice cover is likely to have weakened due to weathering and would have little strength. Spring ice jamming floods for north-south rivers therefore are minor and infrequent. On the other hand, for south-north rivers, the break up starts from the upstream end. The ice floes drifting down the river are likely to be stopped by a rather strong ice cover downstream. Spring ice jamming floods for such rivers therefore are frequent and severe.

2.0 CRITERIA FOR FORMATION OF ICE JAM

2.1 Pariset and Hausser's Criteria

It was mentioned in the last section that as ice floes drift down to an ice cover, their advancement is stopped and the ice floes add to the ice cover. If the flow is slow, the ice floes lay in a single layer. However, if the flow is fast, the ice floes will turn and pile and form a rough fragmented ice cover, or an ice jam.

Pariset and Hausser (1961) proposed that if the leading edge of an ice cover is flooded by the oncoming flow, then a single layer ice cover cannot be formed. The ice floes will be pushed under by the flow and the ice cover can only progress as a rough, fragmented ice cover, or a floating ice jam. Figure 2 shows the leading edge of an ice cover. By writing the Bernoulli equation between the open water and the ice covered parts of the flow, one has

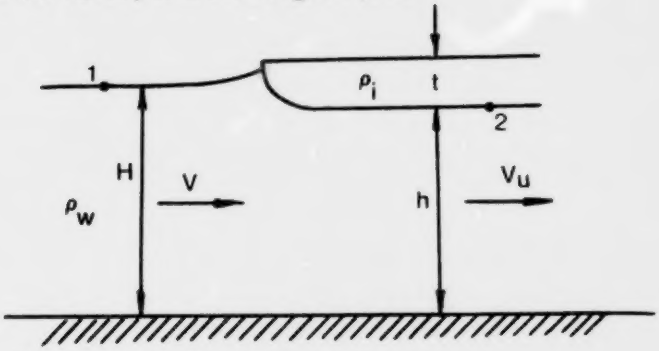


Fig. 2 Leading edge of an ice cover

$$H + \frac{V^2}{2g} = h + \frac{V_u^2}{2g} + \frac{\rho_i}{\rho_w} t \qquad (1)$$

where g is the gravitational acceleration, ρ_i and ρ_w are the density of ice and water respectively and the rest of the symbols are defined in Fig. 2. When the leading edge is flooded, one has

$$H + \frac{V^2}{2g} \geq h + t \qquad (2)$$

From the above two equations and the equation of continuity

$$h\,V_u = H\,V \qquad (3)$$

one obtains

$$F_r = \sqrt{2\frac{\rho_w - \rho_i}{\rho_w}} \left(\frac{t}{H}\right)^{1/2} \left(1 + \frac{1}{2} F_r^2 - \frac{t}{H}\right) \qquad (4)$$

where $F_r = V/\sqrt{gH}$ is the upstream Froude number. The above equation shows that if the upstream Froude number is greater than the right hand side, the ice cover can only progress as an ice jam.

Since F_r is usually less than 0.1 when there is an ice cover, so F_r^2 may be dropped from the above equation because of its small value. This, plus the substitution of $(\rho_w - \rho_i)/\rho_w = 0.08$ into the above equation, reduces it to

$$F_{rc} = 0.4 \left(\frac{t}{H}\right)^{1/2} \left(1 - \frac{t}{H}\right) \qquad (5)$$

where the subscript c means the critical Froude number.

Field observations showed that there is indeed a critical Froude number for ice jamming to occur. When this Froude number is reached, the ice floes at the leading edge will underturn and pile up until it becomes sufficiently thick. Thereafter, the ice cover will propagate upstream with a well-defined minimum thickness.

There is another Froude number for the leading edge of an ice jam to become unstable. Pariset and Hausser gave the empirical value of such a Froude number as 0.158. When $F_r > 0.158$, the leading edge of the ice cover can progress no more, not even as an ice jam. Instead, the oncoming floes will go under the ice cover and form an underwater hanging dam. The ice dam increases the upstream stage and reduces the upstream velocity. The net effect is to reduce F_r to less than 0.158, under which condition, the ice cover can propagate once again.

2.2 Michel's Criteria

Michel (1971) also studied the stability of the leading edge of the ice cover. Instead of writing the Bernoulli equation in the form of Equation (1), he included the pressure terms in the equation also and wrote

$$\frac{p_1}{\rho_w g} + \frac{V^2}{2g} + H = \frac{p_2}{\rho_w g} + \frac{V_u^2}{2g} + h \qquad (6)$$

where p is the pressure and the subscripts 1 and 2 denote the two points shown in Figure 2. From the above equation, by rearrangement, one obtains

$$\frac{p_1 - p_2}{\rho_w g} = \frac{\Delta p}{\rho_w g} = \frac{1}{2g}\left(V_u^2 - V^2\right) - (H - h) \qquad (7)$$

Michel proposed that when the pressure differential Δp is large enough as to overcome the buoyancy, the ice will be pulled down and form an ice jam. Mathematically, this means

$$\Delta p \geq (\rho_w - \rho_i)\, g t = \Delta\rho g t \qquad (8)$$

as the ice jamming criterium.

From Equations (7), (8), (3) and

$$H - h = \frac{\rho_i}{\rho_w} t \qquad (9)$$

Michel obtained the following equation

$$F_r \geq 0.4 \left[\frac{t}{H} \cdot \frac{1 + \frac{\rho_i}{\Delta\rho}\left(1 - \frac{\rho_i}{\rho_w}\frac{t}{H}\right)^2}{1 - \left(1 - \frac{\rho_i}{\rho_w}\frac{t}{H}\right)^2} \right]^{1/2} \qquad (10)$$

as the governing equation for the initiation of ice jamming in an ice cover. The above equation may be compared with Pariset and Hausser's equation, i.e., Equation (5) in 2.1.

2.3 Uzuner and Kennedy's Criteria

Uzuner and Kennedy (1972) considered that the stability of the leading edge of an ice cover is reflected by the stability of the ice floes at the leading edge. Fig. 3 shows such an ice floe resting against a solid ice cover. The high flow velocity under the floe causes the floe to dip. On the floe, there act three moments about the downstream, lower edge of the floe:

1. a clockwise moment produced by the pressure under the ice floe.
2. An anti-clockwise moment produced by the weight of the floe, and
3. Another anti-clockwise moment which is produced by the negative pressure under the ice cover as a consequence of flow separation at the upstream, lower edge of the floe.

The pressure producing moment 1 and the pressure producing moment 3 really are not separable. They are separated theoretically only for easy mathematical modelling.

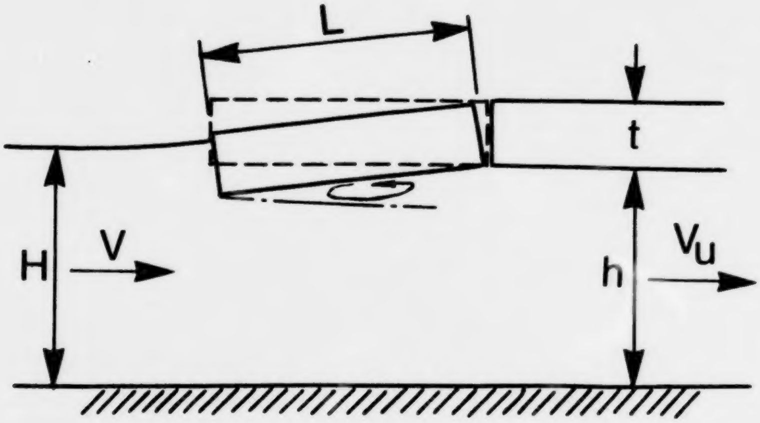


Fig. 3 Stability of ice floe at leading edge of ice cover

Uzuner and Kennedy proposed that if the equilibrium of the three moments can only be reached with the upstream upper edge dipped under water, then the ice floe will not be at equilibrium but be pushed down by the flow and cause ice jamming. From the equation of the moments, they found that the critical Froude number for ice jamming is given by the following equation:

$$F_{rc} = f\left(\frac{\rho_i}{\rho_w}, \frac{t}{H}, \frac{t}{L}, C_m, C_s\right) \tag{11}$$

where C_m is a multiplying coefficient for the moment due to flow separation and C_s is another coefficient accounting for different values of the average flow velocity and surface flow velocity. For a logarithmic velocity distribution, the value of C_s is 1.30. By performing a large number of laboratory experiments and analysing the data, they found C_m is a function of ρ_i/ρ_w and t/L. Equation (11), thus, is reduced to

$$F_{rc} = f\left(\frac{\rho_i}{\rho_w}, \frac{t}{H}, \frac{t}{L}\right) \tag{12}$$

Fig. 4 shows their experimental results for floes of a material of ρ_i/ρ_w=0.67. The important conclusion from Fig. 4 is that both thin and thick ice floes are less stable than floes of medium thickness. The break up pattern and the freeze up history thus have a great deal of influence on the jamming of ice in the spring in a river.

Fig. 5 shows the experimental data for ice. In the figure, the theoretical curve by Pariset and Hausser is also shown for comparison. From Fig. 5, one sees that when the value of $F_{rc}/\sqrt{2}$ is 0.1, all ice covers will jam. This corresponds to a critical Froude number of 0.141, very close to the value of 0.158 as suggested by Pariset and Hausser.

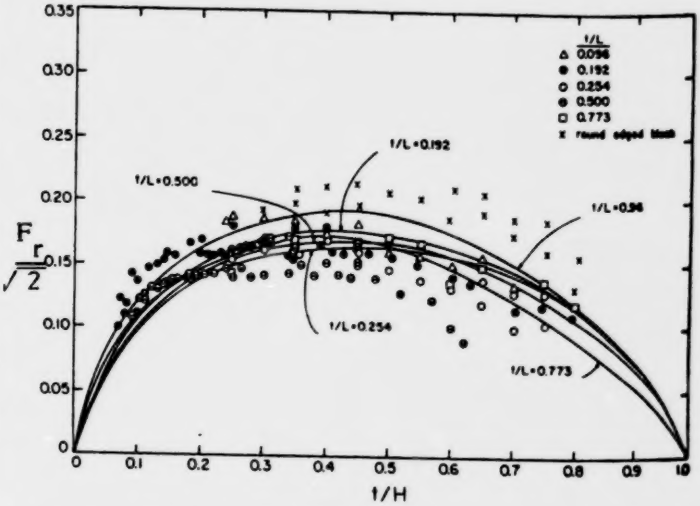


Fig. 4 Experimental results by Uzuner and Kennedy for ρ_i/ρ_w=0.67

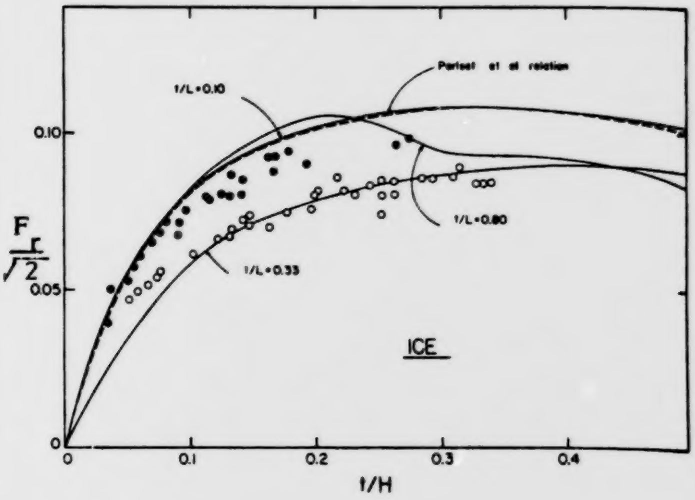


Fig. 5 Experimental results for ice

Ashton (1974) used the same approach but simplified the momentum equations and was able to obtain the following explicit equation for the critical Froude number for ice jam:

$$F_{rc} = 2\left(1 - \frac{t}{H}\right)\left[\frac{\frac{t}{H}\left(1 - \frac{\rho_i}{\rho_w}\right)}{5 - 3\left(1 - \frac{t}{H}\right)^2}\right]^{1/2} \tag{13}$$

He fit the above equation to the data by Uzuner and Kennedy and showed that the fitting was better than the equation by Pariset and Hausser.

2.4 Jamming of Ice at Bridge Piers

It was mentioned earlier that when the flow of ice is stopped by constrictions, including bridge piers, ice will jam up in front of it and cause ice jamming floods.

Fig. 6 shows ice floes discharging down a river and coming to a bridge.

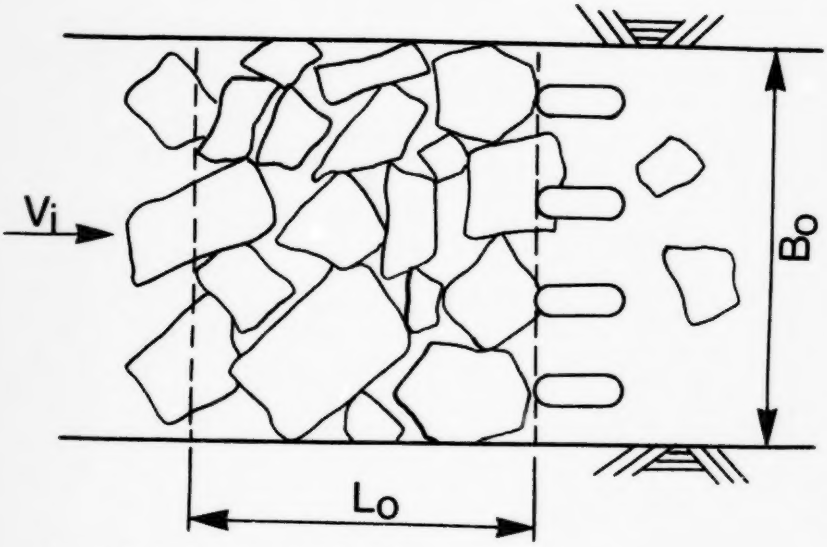


Fig. 6 Ice field in front of bridge piers

From river ice mechanics, it is learned that only a small stretch of the ice field in front of the bridge transmits its load to the bridge piers. Beyond this stretch, the load is taken up by the banks. Let the length of the effective stretch be L_o, the kinetic energy of the ice field will be

$$E_k = \frac{1}{2}\left[L_o B_o t \rho_i\right] V_i^2 C_i \qquad (14)$$

where C_i is the percentage of surface ice coverage and V_i is the velocity of the ice field. If the bridge has n piers, then the kinetic energy of the ice field transmitted to each pier will be 1/n of the above value.

Korzhavin (1962) proposed that if the kinetic energy transmitted to each pier is large enough to destroy an ice floe, then the pier will cut through the floe and the ice will not jam.

If the longitudinal dimension of an ice floe is L and the force for cutting through it is F, the condition for ice-jamming free bridge pier design will be

$$\frac{(L_o B_o t \rho_i) V_i^2 C_i}{2n} \geq FL \qquad (15)$$

If the above inequality is satisfied, ice jams will not develop in front of a bridge.

There is much theory to evaluate or to estimate the parameters L_o, V_i, C_i, L and F. However, this is beyond the scope of the present paper and will not be discussed here.

3.0 RESISTANCE OF ICE JAMS

Once an ice jam is developed, the local resistance is increased. Facing an increased resistance, the upstream stage must be increased in order to pass the same rate of discharge under the ice jam. This, in fact, is the reason for upstream flooding.

It was mentioned in section 1 that an ice jam can be floating or dry and that the latter is mostly responsible for severe spring ice jamming floods. Important as it is, there is no compiled information of the resistance of dry ice jams.

For a floating ice jam, it can be either static or dynamic, and an underhanging dam may or may not be under the ice jam. Again, important as dynamic ice jams and underhanging dams are, there is no documented information on their resistance to the flow.

The only existing documented information and theories are for the resistance of static floating ice jams in the absence of hanging dams. Even in this case, the theories and supporting data can hardly be considered as satisfactory.

Thus far, the major effort has been on how to evaluate the overall, or the composite Manning's n of a river covered by a floating jam, or a fragmented ice cover.

The study of the Manning's n of an ice covered river is along the same line as the study of the composite roughness of a river having different boundary roughnesses. Fig. 7 shows the cross section of such a river. If the cross section is divided into subsections according to the boundary roughnesses, one can denote the wetted perimeter and the Manning's n of the subsections by $p_1, p_2, \ldots p_N$ and $n_1, n_2 \ldots n_N$. Following the subdivision of the cross section, different conditions may be used to obtain formulae to calculate the composite Manning's n of the river. Fig. 8 shows three such formulae (above the dash-dotted line) and the conditions used for their derivation.

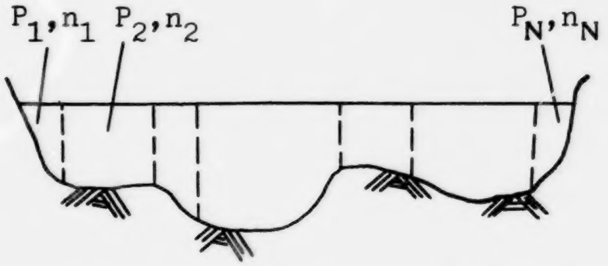


Fig. 7 Subdivision of cross section of different roughnesses

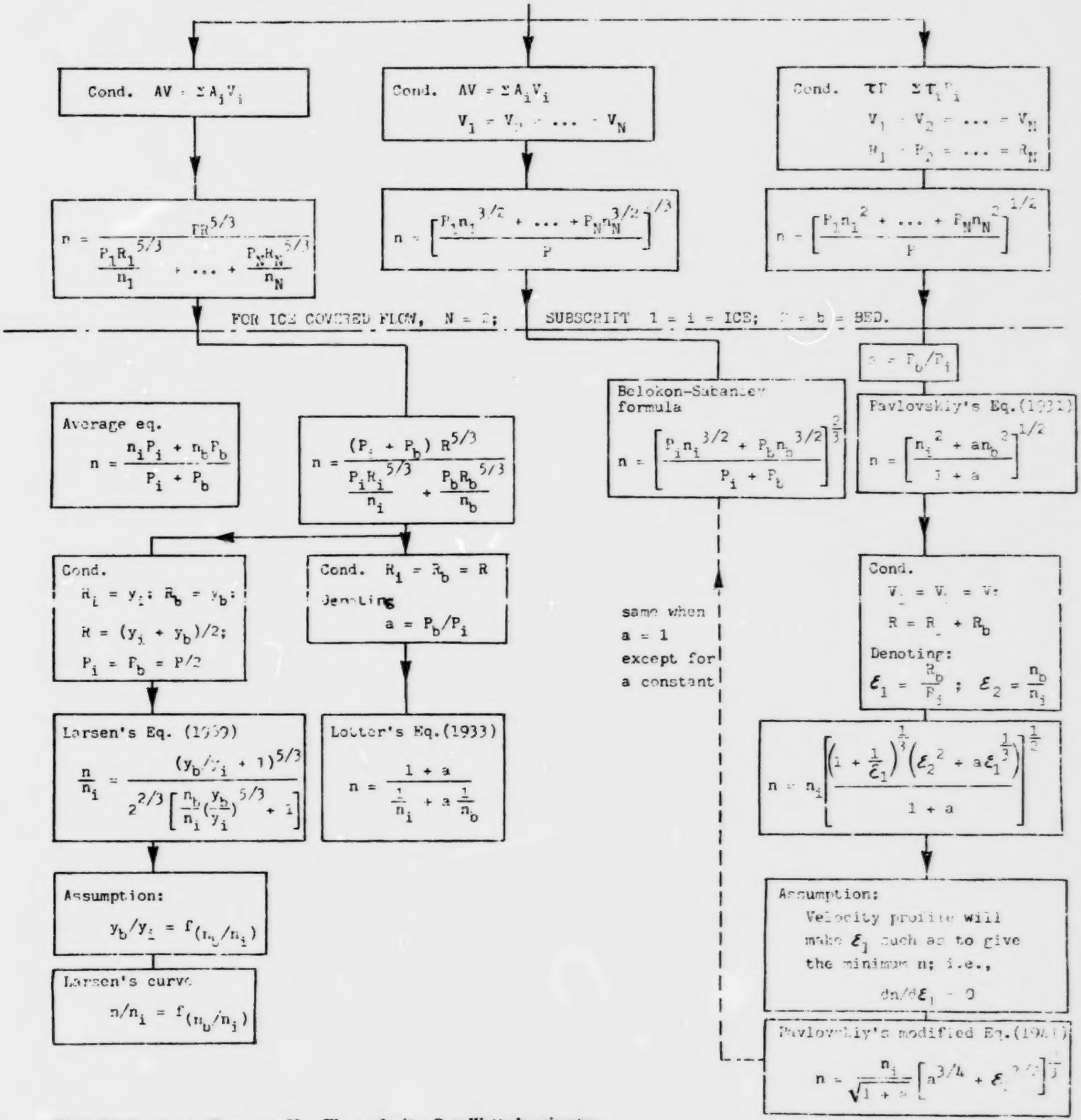
For a river under an ice cover, it may be divided into two parts only; the upper part that is affected by the ice cover alone and the lower part that is affected by the river bottom alone. With such a consideration, the equations for calculating the composite Manning's n are reduced to those below the dash-dotted line in Fig. 8.

Fig. 8 also shows in detail how the better known equations, such as the Larsen equation; the Lotter equation; the Belokon-Sabaneev formula; the Pavlovskiy equation and the modified Pavlovskiy equation are derived from the above three equations and the additional assumptions or constraints used. Based on Fig. 8, one sees that once the Manning's n for the ice jam and the Manning's n for the river bed are known, there are more than enough formulae on hand to calculate the composite Manning's n for the ice jam.

One sees from Fig. 8 that the formulae for calculating the composite Manning's n are far too many and that many of the assumptions are far too arbitrary. The problem with the large number of formulae is further complicated by the fact that all the equations seem to work because in practice it is usually the Manning's n of the river bed and the Manning's n of the whole ice-covered river that are measured and from them the Manning's n of the ice cover is subsequently calculated. Thus, depending on the equation used, different Manning's n values for the same ice cover may be obtained. The resubstitution of the Manning's n for the ice cover or the ice jam so obtained back into the equation naturally will give the correct composite roughness - the composite Manning's n measured.

Although Manning's n for ice jams may be evaluated by different formulae. This fact, however, is seldom pointed out in publications listing the

COMPOSITE ROUGHNESS OF CHANNELS OF NON-UNIFORM BOUNDARY ROUGHNESSES


Cond. $AV = \Sigma A_i V_i$
Cond. $AV = \Sigma A_i V_i$
$V_1 = V_2 = \ldots = V_N$
Cond. $\tau P = \Sigma \tau_i P_i$
$V_1 = V_2 = \ldots = V_N$
$R_1 = R_2 = \ldots = R_N$
$n = \frac{PR^{5/3}}{\frac{P_1R_1^{5/3}}{n_1} + \ldots + \frac{P_NR_N^{5/3}}{n_N}}$
$n = \left[\frac{P_1n_1^{3/2} + \ldots + P_Nn_N^{3/2}}{P}\right]^{2/3}$
$n = \left[\frac{P_1n_1^2 + \ldots + P_Nn_N^2}{P}\right]^{1/2}$
FOR ICE COVERED FLOW, N = 2; SUBSCRIPT 1 = i = ICE; 2 = b = BED.
$a = P_b/P_i$
Average eq.
$n = \frac{n_iP_i + n_bP_b}{P_i + P_b}$
$n = \frac{(P_i + P_b)R^{5/3}}{\frac{P_iR_i^{5/3}}{n_i} + \frac{P_bR_b^{5/3}}{n_b}}$
Belokon-Sabaneev formula
$n = \left[\frac{P_in_i^{3/2} + P_bn_b^{3/2}}{P_i + P_b}\right]^{\frac{2}{3}}$
Pavlovskiy's Eq.(1931)
$n = \left[\frac{n_i^2 + an_b^2}{1 + a}\right]^{1/2}$
Cond.
$R_i = y_i$; $R_b = y_b$;
$R = (y_i + y_b)/2$;
$P_i = P_b = P/2$
Cond. $R_i = R_b = R$
Denoting
$a = P_b/P_i$
same when
a = 1
except for
a constant
Cond.
$V_i = V_b = V$
$R = R_i + R_b$
Denoting:
$\varepsilon_1 = \frac{R_b}{R_i}$; $\varepsilon_2 = \frac{n_b}{n_i}$
Larsen's Eq. (1969)
$\frac{n}{n_i} = \frac{(y_b/y_i + 1)^{5/3}}{2^{2/3}\left[\frac{n_b}{n_i}\left(\frac{y_b}{y_i}\right)^{5/3} + 1\right]}$
Lotter's Eq.(1933)
$n = \frac{1 + a}{\frac{1}{n_i} + a\frac{1}{n_b}}$
$n = n_i\left[\frac{\left(1 + \frac{1}{\varepsilon_1}\right)^{\frac{1}{3}}\left(\varepsilon_2^2 + a\varepsilon_1^{\frac{1}{3}}\right)}{1 + a}\right]^{\frac{1}{2}}$
Assumption:
$y_b/y_i = f_{(n_b/n_i)}$
Larsen's curve
$n/n_i = f_{(n_b/n_i)}$
Assumption:
Velocity profile will
make ε_1 such as to give
the minimum n; i.e.,
$dn/d\varepsilon_1 = 0$
Pavlovskiy's modified Eq.(19[illegible])
$n = \frac{n_i}{\sqrt{1 + a}}\left[a^{3/4} + \varepsilon_2^{3/2}\right]^{\frac{2}{3}}$

SYMBOLS: A = Flow area; V = Flow velocity; P = Wetted perimeter;
R = Hydraulic radius; n = Manning's roughness coefficient;
y = Depth of flow; τ = Shear stress at the boundary

Figure 8. Formulae for Calculating Composite Manning's n and Their Derivation

Manning's n values. One of the most important tasks that one has to do thus is to reduce the number of formulae and to convert all the Manning's n values of the ice cover to the same basis.

Nezhikhoviskiy (1964) published the following table for the Manning's n of ice jams. The table was derived from river data in USSR. From Table 1, one sees that n_i increases with the thickness of the ice jam and that the n_i for a jam of ice floes is higher than the n_i for a jam of slush of the same thickness.

Table 1 Manning's n (n_i) for ice jams

Thickness of Jam (m)	Loose Slush	Dense Slush	Ice Floes
0.1	-	-	0.015
0.3	0.010	0.013	0.04
0.5	0.01	0.02	0.05
0.7	0.02	0.03	0.06
1.0	0.03	0.04	0.07
1.5	0.04	0.06	0.08
2.0	0.04	0.07	0.09
3.0	0.05	0.08	0.10
5.0	0.06	0.09	-

The above table shows that a thicker jam is a rougher jam. Although Table 1 appears to be a little crude, it is the only available published data today. It may also be pointed out that the above table was deducted from river data collected during the winter freeze-up period and the depth of the rivers was typically 2-3 m.

In Canada, n_i values of 0.05-0.06 have been obtained by Pariset et al (1966) for the St. Lawrence River and 0.095 to 0.10 by Beltaos (1979) for the Smoky River.

4.0 CONCLUSIONS AND DISCUSSIONS

From the preceding section, one sees that, scientifically, the problem of ice jamming and ice jamming flood has hardly been solved. Although one is now in the position to predict the occurrence of ice jamming with certain confidence, what happens to the ice cover and what its resistance to flow will be once it becomes unstable remain in the dark.

Limited understanding on the resistance of an ice jam thus far is only confined to static floating jams; and, even for that, there is much room for theoretical improvement and data acquisition. For dynamic jams, dry jams or underhanging dams, the present knowledge about their evolution and resistance to flow is practically nil. Since major ice jamming floods occur when a static jam becomes unstable and results in underhanging dams or dry jams, the study of the dynamic evolution of dry jams and hanging dams and the accompanying resistance characteristics, therefore, should receive priority in future research.

Cold weather hydraulics research has not received its rightful share of attention in Canada in the past. The situation has not improved much today. Researchers in the area are too few in number and spread too thin. A viable group of several scientists and engineers should be formed so problems on cold weather hydraulics, ice jamming floods among them, can be solved effectively and collectively. Better direct liaison between working level field engineers and scientists should also be established for closer and more effective cooperation.

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WORKING GROUP 2

Hydrologic Modelling

GROUPE DE TRAVAIL N^{o} 2

Modélisation hydrologique

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Modélisation hydrologique

METEOROLOGICAL INPUTS
FOR HYDROLOGICAL
MODELS

prepared for:

The IWD Technical Workshop on
Modelling Activities Related
to Flood Damage Reduction

May 23-25, 1979
Ottawa

by: R.G. Lawford
Chief, Scientific Services Division
AES, Ontario Region

1. Introduction

It is well known that precipitation and, during the spring, temperature are the principal controls of the hydrological cycle. With the exception of floods caused by ice jams, damaging floods can usually be traced to a combination of these two factors. It is natural then that these two parameters and related factors, such as soil moisture and evaporation, should play very important roles in many flood models. As a result, the reliability of the outputs from these models will be strongly influenced by the reliability of meteorological observations and forecasts.

This paper does not directly address all the points raised in the directions to authors. However, it does provide:

- a review of the historical and real-time data available from AES,
- a review of the forecasts of precipitation and temperature available from Regional Weather Centres and from the Canadian Meteorological Centre,
- a discussion of new technologies and how data from these systems could be incorporated into hydrological models,
- a brief outline of changes that could be made to hydrological models, such as the SSARR[1] model, to provide more realistic representations of the meteorological processes,
- recommendations for further action on hydrological models.

2. Historic and Real-Time Meteorological Data

Meteorological observations are important to hydrological modellers for two main reasons. First, historical records are required to calibrate a hydrological model. Second, real-time precipitation and temperature measurements are useful in estimating the water currently running off into the streams and, in some cases, monitoring model performance. In addition, meteorological data are useful in specifying initial conditions of soil moisture and snow on the ground.

Observational networks are managed by the AES Regional Headquarters. There are two basic networks: one which provide real-time hourly observations and another which provides twice-daily observations at the end of each month.

The first network, referred to as the synoptic network, consists of 318 stations across Canada. Hourly (or in some cases 3-hourly) measurements of temperature and 6-hour accumulations of precipitation are recorded at these stations and then transmitted via teletype of central weather offices and the Canadian Meteorological Centre computer in Montreal.

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[1] -SSARR- refers to the U.S. Corps of Engineers Streamflow Resevoir Regulation Model.

The second network consists of 1797 stations across Canada which record temperature and precipitation and an additional 403 stations that report precipitation only. The stations are operated by volunteers and contract observers who take readings at 0800 and 1700 LST. These data are recorded on a form and, at the end of the month, the form is forwarded to regional Scientific Services Climate Divisions where they are quality controlled and copies and summaries are disseminated to users. It is possible to make special arrangements with some of these observers whereby the flood control agencies can call the observers directly on a daily basis to get precipitation data. The data are also sent to the Canadian Climate Centre (CCC) for further checking and for entry into the national data archive. Magnetic tape copies, computer analyses of data and national publications of the data are available through the CCC.

Neither of the above networks was designed specifically for hydrological models. Consequently, it is useful to determine the representativeness of data from these stations and the uncertainties that result from using them to estimate the water loading on a watershed. According to studies in the U.S. carried out for 3 or 4 day storms, the standard error of estimate is 6% for a gauge density of 100 mi^2 per gauge and 14% for a density of 500 mi^2 per gauge (Linsley et al, 1949). As the averaging time decreases for a given gauge density, the accuracy of areal estimates also decrease. Huff and Neil (1957) showed that for rainfalls in areas characterized by frequent thunderstorm activity a 100 mi^2 per gauge network density gave a standard error of estimate of 25%. Therefore, we can conclude that higher density raingauge networks will be required for reliable areal precipitation estimates in areas where the ratio of thunderstorm rainfall to synoptic storm rainfall is high.

Another method of assessing the representativeness of point rainfall measurements has been used by Mapanao (1979) in the Ottawa River basin. Mapanao calculated inter-station correlations for daily precipitation amounts. According to his results, the correlation coefficient relating rain at Maniwaki, the most representative station in the basin, and rain at Ottawa is only 0.42 (for distance of only 90 miles). This result suggests that uncertainties will be very large for parts of Canada where convective storms or local effects produce heavy increases in rainfall. There is a requirement to assess the network density across Canada for precipitation and temperature taking into account variations in precipitation due to varying climatic regimes and possible influences of topography on climate. However, it must be recognized that the resouces will likely not be available to make major increases in the density of stations in the observing networks. Consequently, there is a need to develop new techniques for estimating areal distributions of temperature and accumulated amounts of rainfall from the existing networks.

3. AES Precipitation and Temperature Forecasts

3.1. Short-Duration Forecasts

3.1.1. Quantitative Precipitation Forecasts (QPF)

Predicted values of rainfall amounts are even less detailed than the observed amounts. The principal tool used in the preparation of quantitative precipitation forecasts is referred to as the QPF model. The model produces predictions of precipitation accumulations for four 12 hour intervals at points on a grid with a spacing of 190 km. The value shown at the point is actually an average for the square grid area surrounding the point.

The preparation of the QPF forecast is actually a two-step process. First, the observed data are entered into a numerical predictive model that integrates the equations governing atmospheric motions along with thermodynamic and moisture equations. The outputs from the predictive model include stream functions for a number of levels of the atmosphere and estimates of terrain-induced vertical velocities. The second step involves entering these forecast values into the diagnostic QPF model which uses them to determine the moisture, horizontal wind, vertical velocity, temperature and dewpoint depression fields for each 12-hour period. The QPF model then derives large-scale and small-scale precipitation forecasts for each 12-hour period.

The model handles large scale storms very well but encounters difficulty in forecasting smaller scale convective activity. Showers are incorporated into the model by paramaterizations which relate precipitation amount to temperature gradients, atmospheric instability and day-time heating.

Since the model is diagnostic, there are two types of errors that limit its accuracy. First, there are errors in the atmospheric states predicted by numerical prediction models. Second, there are errors resulting from the limitations of the diagnostic model itself. Although verification studies have tended to stress the correctness of the area for which precipitation was forecast rather than the amount, studies have found that the forecast amounts are underestimated when a large proportion of the precipitation is produced by convective clouds. Futhermore, these errors increase rapidly in time.

Outputs from the QPF model are presented on maps and transmitted via facsimile to Regional Weather Centres at 1000 and 2000 EST. Maps are prepared for 0-12, 12-24, 24-36 and 36-48 hours from the initial time. Grid point values of these 12-hour precipitation amounts are transmitted on teletype. In addition, the probabilities of precipitation on each of days 3, 4 and 5 are computed using an analogue forecast technique.

It must be stressed that these grid point values do not represent the precipitation at a specific point. Therefore, these values must be adjusted to include the effects of terrain, lakes, preferred locations for storm development, storm trajectories, etc. Normally this is done by meteorologists located at Regional Weather Centres. In the case of large basins, climatological studies can be carried out to compute the most probable spatial distribution of rain given a single value forecast rainfall amount for the area.

3.1.2 Regional Weather Centre Outputs

Regional Weather Centres prepare and distribute maximum and minimum temperature forecasts for the next 5 days along with forecast precipitation amounts for the next two days. At the present, in Ontario, public forecasts for days 1 and 2 are issued at 0505 and 1700 EST. The 5-day forecast is issued at 0545 EST. According to studies carried out at the Ontario Weather Centre, the subjective input to the preparation of extended range temperature forecasts improves their quality. The proportion of useful temperature forecasts decreases from 70 to 85% on day 3 to 55 to 65% by day 5.

3.2 Special Forecast Services

Special projects have been carried out in the Saskatchewan River Basin, the Saint John River Basin and the Ottawa River Basin. In each case the Atmospheric Environment Service through the Canadian Climate Centre's Hydrometeorology Division and Regional offices, has provided special meteorological support. In the case of the Ottawa River Forecast system the Quebec Weather Centre provides forecasts of precipitation amounts for the first 36 hours. For days 3, 4 and 5, precipitation probabilities were categorized as being either 1 or 0 for the occurrence or non occurrence of precipitation. If the value was 1 then a precipitation amount, derived from all the days with rain for that date, was assigned. Beyond day 5 average values of temperature and precipitation amounts were entered into the model.

Pugsley (1977) verified the forecasts that were being provided to the Ottawa River flood forecasting system. He found that the ability to produce correct point and basin precipitation forecasts decreased after day 1. However, on the average for the 4 months verified, the quantities forecast for all 5 days were within 3 mm/day of the observed amount. In three of the four months, more than 60% of the event forecasts for the first 4 days were correct. In addition, temperature forecasts for days 1 and 2 were, on the average, within 2°C of the correct value. The average absolute error increased to 4°C by day 5.

3.3 Limits to Forecasting

Sanders (1973) evaluated the ability of MIT meteorology staff and students to forecast temperature and probability of precipitation. According to his report, their skill in forecasting temperatures dropped to 10% in 4 days while their skill in forecasting probability of precipitation decreased to 10% in 2½ days.

The American Meteorological Society has recently released a policy statement on weather forecasting stating that "Daily temperature forecasts of moderate skill and usefulness are possible (up to 5 days). Forecasts of precipitation amounts at an equivalent level of skill can be made to 3 days. The precipitation skill decreases through the fourth and fifth days but maintains a slight margin over climatology."

On a more optimistic note, theoretical studies indicate that the upper limit for useful daily forecasts is between 1 and 2 weeks, depending on the criteria used to define 'useful'.

3.4 Long-Range Forecasts

The 15-day temperature anomaly forecast chart is the only official long-range forecast currently being issued by the AES. The forecasts are prepared at the Canadian Meteorological Centre by an analogue technique which prepares forecast values for 70 points. The forecasts are categorized as normal, above or below normal (temperature anomaly less than .84 X standard deviation (SD) but more than .2 X SD) and much above or much below normal (temperature anomaly greater than .84 X SD). The map is issued once a week. It is transmitted by facsimile to Regional Weather Centres and published in "Climatic Perspectives".[2] According to Robinson (1976):

- If the forecast temperature anomaly is much above (much below) normal, then the actual temperatures will be much above (much below) or above (below) normal 75% of the time;
- If the forecast temperature anomaly is above (below) normal, then the actual temperatures will be much above (much below) or above (below) normal 60% of the time;
- Forecasts of normal conditions verify 50% of the time.

Various divisions within the AES also carry out activities related to long-range forecasting. At present, the Monitoring and Prediction Division of the CCC is evaluating methods for predicting monthly and seasonal temperature and precipitation anomalies. It is also possible that probability forecasts, similar to those produced in the U.S. for monthly precipitation anomalies exceeding certain thresholds, could be prepared in Canada. The significant thresholds could be adapted to the needs of water supply and even flood forecasting on a seasonal or a monthly basis. Among the most promising techniques are those which rely on sea surface temperature anomalies.

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[2] Climatic Perspectives is a weekly publication of the Canadian Climate Centre.

The Hydrometeorology Division of the CCC has used Hovmoller diagrams to predict the timing and general location of major storm development. These diagrams facilitate the analysis of wave motions at a height of 50.0 kPa (approx. 6000 m). On the basis of Mapanao's experience (1979), it is possible to project these quasi-conservative features 1 to 2 weeks into the future in order to provide a preliminary assessment of the time and longitudes where major storm development can be expected.

4. New Technologies

4.1 Weather Radar

During the past decade numerous advances have been made in the processing and delivery of quantitative information from weather radar. Presently, the AES supports 6 radar/computer (SCEPTRE) systems that are capable of providing real-time facsimile maps showing the distribution of precipitation rates and, on a historical basis, magnetic tapes containing quantitative values of derived rainfall intensities. The principal real-time outputs are maps showing 8 discrete intensity ranges of reflectivities from signals back-scattered from the precipitation at constant altitudes (CAPPI maps) as well as maps showing echo tops and the maximum reflectivity in the storm.

Weather radars emit and detect radiation in the range from 5450 and 5820 MHz. The spatial resolution for CAPPI maps is 2 km X 2 km. The time separation between maps is approximately 6 minutes. Development work is currently being undertaken to:

- provide data for CAPPI maps over standard dedicated telephone lines,
- provide a dial-in service so users can select the types of output that they desire.

Marshall et al (1955) and others have developed formulae for relating reflectivities in low levels (1 km) and rainfall intensities. These relationships vary according to the type of rain or other precipitation that is being observed. By integrating intensities at each point over a time interval it is possible to derive a map of rainfall. These maps would provide much more detailed rainfall estimates than does our current raingauge network. According to Harold et al (1973), radar estimates give the same accuracy for a convective storm as does a raingauge network of a density of one per 25 km^2. On the other hand, radar gives an accuracy equivalent to one gauge per 100 km^2 for a widespread rain. Another advantage to radar observations lies in their potential to provide rainfall accumulations on sub-basins. Gorrie and Kouwens (1977) have demonstrated the feasibility of using this procedure in the Grand River Basin.

Currently a technique referred to as SHARP (Short Range Automated Radar Prediction) is being developed at McGill University for predicting the distribution of precipitation using the shape and intensity of the existing radar echo. By correlating

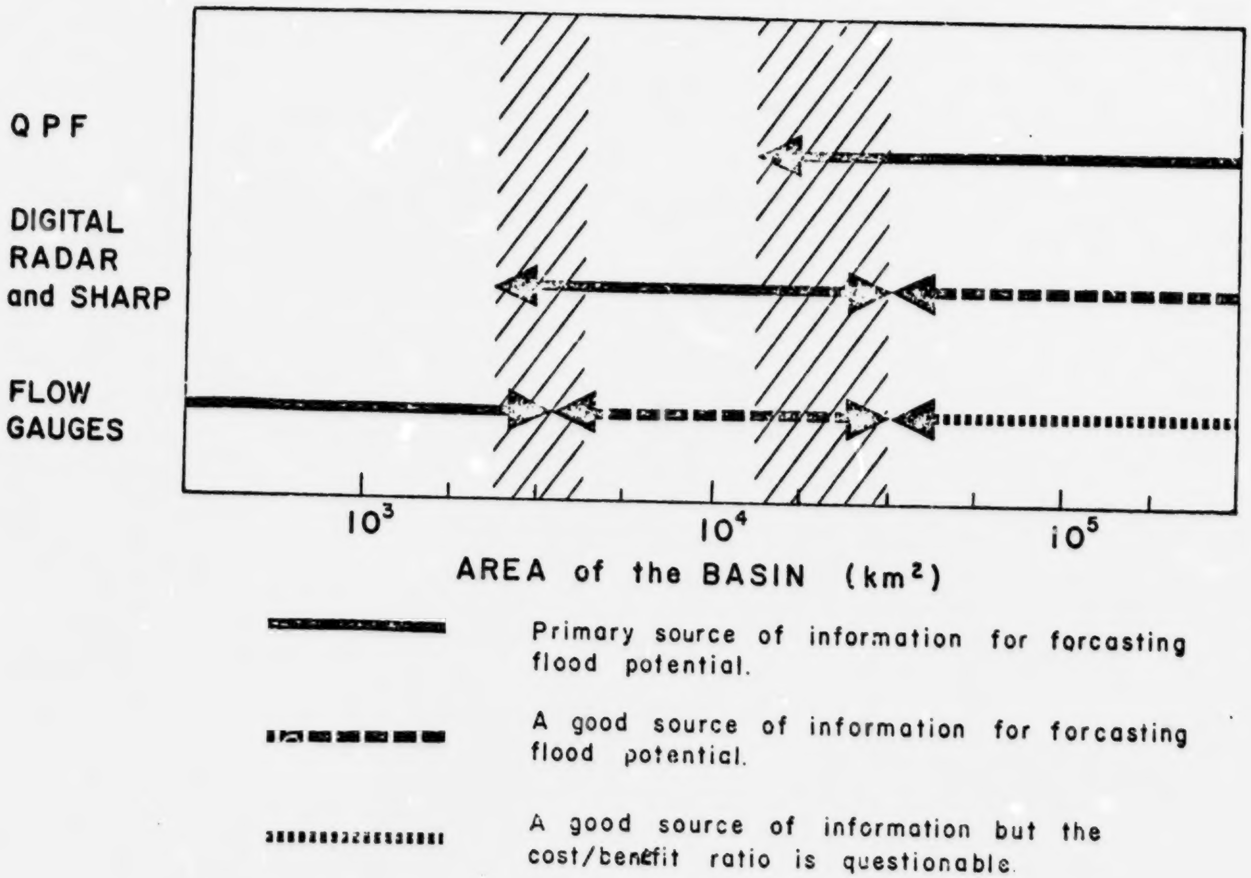

Figure 1. Graph showing the types of observations and forecasts required for watersheds of various sizes.

the present shape of the echo with its shape at an earlier time it is possible to derive a regression equation that will predict the distribution of precipitation as much as 6 hours ahead. By combining this predictive technique with the methods of deriving rainfall amounts from radar described in the previous paragraph, it will be possible to have a highly detailed 3-6 hour forecast of precipitation accumulations.

Unfortunately, SCEPTRE systems are expensive. The present cost is upwards of $1,000,000. The question of cost effectiveness arises. Figure 1 can be used as a very rough guide for determining types of observations and predictions needed for watersheds of various sizes. Although this figure is not the final solution, it could serve as a basis for discussions that could lead us to a final answer.

4.2 Satellites

Satellite imagery can also be used to provide valuable inputs for flood forecast models. A number of studies have demonstrated the utility of satellite information in estimating rainfall rates and the extent of snow cover. It is also possible that satellite imagery may be assistance in estimating snow depths and snow pack densities.

Satellite imagery is already being used in the U.S.A. and in the Saint John River Basin. In both areas infra-red imagery recorded by the TIROS-N is being used to derive the area of snow cover once a week (cloud conditions permitting). The infra-red imagery can be used to estimate an average surface temperature for a square 2 km by 2 km. The temperature gradient is analyzed and the boundary of the snow cover is placed on the warm side of the strong gradient. Since the coordinates of each sub-basin are known, it is relatively simple to sum up the number of coordinate points that are within the snow cover area and to arrive at an estimate of the percentage of the surface covered by snow. These proportions can then be entered into a hydrological model to provide better snow-melt estimates.

Satellites can also be used as aids in communications. It is possible to have telemetering flow and rain gauges transmit their data to a GOES satellite. The data are then relayed back to Wallops Island, Virginia. From there, it can be acquired either through a computer to computer link or, in cases where AES has an interest in the data, via AES weather circuits.

5. Questions Regarding the Meteorology of Hydrological Models

Although I have not had extensive exposure to hydrological models, I do have a few questions about them. Perhaps some of the following questions can be topics for further discussion.

- Would it be possible to use a better procedure for incorporating evaporation into hydrological models? The Ontario Weather Centre prepares a drying index for agricultural forecasts that includes temperature, wind speed, sunshine hours and vapour pressure deficits. Although the absolute day to day variations in evaporation may be relatively small in the spring, they are very significant during the summer months. Is it possible that a new formulation using more meteorological inputs rather than the current practise of using average daily evaporation amounts would produce significant improvements in the results?

- Can the applications of the model outputs be used to determine the best types of inputs for hydrological models? At present, mean rainfall amounts are being used as input beyond day 5. For example, if the model is being used to assess flood potential would it not be better to use another rainfall amount such as the average rainfall on a day with rain?

- Given the uncertainties in the forcasts of meteorological parameters used in hydrological models, would it not be desirable to have models that would predict a range of flows rather than single value outputs? For example, using the range of rainfall amounts expected over a sub-basin in the area, it might be possible to determine the probability of flooding conditions. Manual overides could also be added to existing models to allow for the entry of different forecast sequences and amounts of precipitation. It is possible to develop hydrological models that work backward {ie. starting with the flows that would produce flooding conditions, the models would determine how much rain would have to fall (at normal temperatures) to produce these conditons}. Based on this value the Regional Weather Centre would be requested to specify the probability of rainfalls exceeding this amount.

6. Recommendations

Probably, given sufficient resources and time, a joint IWD/AES project could be mounted to develop a totally new improved flood-damage warning system. Unfortunately, at present, neither the resources nor the time are available. However, the following steps could be taken to improve meteorological inputs to hydrological models.

6.1 IWD, in concert with AES, should conduct detailed sensitivity analyses for each hydrological model that is being used operationally. These analyses should indicate the level of error that occurs in the final predictions of flow based on uncertainties in precipitation measurements and forecasts, temperature forecasts, the area of snow cover and evaporation estimates. The effects of uncertainties in these parameters would be analyzed individually and in combination. The analyses should also be carried out for spring and summer conditions and for watersheds with differing dimensions and topographical features. Emphasis should be placed on the biases and errors most frequently found in meteorological observations and forecasts and in the interpolation schemes that are applied to meteorological data.

The resulting sensitivity analyses would be used to priorize modifications to hydrological models, to justify projects designed to improve the accuracy of precipitation forecasts, to justify through cost/benefit studies and to design better observational networks and to assess the benefit-to-cost ratios associated with the implementation of new radar and satellite technologies. In this connection it would be helpful to have a hydrologist specify the level of accuracy that is needed in a flood forecast and then to have this level related to the accuracy required in the areal estimates of observed and forecast precipitation.

6.2 In view of the fact that the resources for increasing station densities in meteorological networks are not likely to increase, AES should develop and test improved methods for estimating areal averages of precipitation amount and temperature from the existing networks for all data-sparse regions of Canada. At present most hydrological models rely on simple weighing techniques to compute the average. However, more sophisticated objective analyses schemes are available for use in deriving areal averages. After the AES has developed improved equations for estimating areal averages these should be incorporated into hydrological models.

6.3 AES should increase its efforts in developing automated procedures for deriving rainfall amounts for selected periods from radar data and for preparing estimates of the percentage snow cover in catchements from satellite imagery. In addition, there is a requirement for better methods for delivering outputs from these new systems to flood forecast centres and for policies to regulate the provision of these new services. Although computer to computer links seem most feasible, there is a need for AES to explore all the options (for example, could the flood forecast model and the meteorological inputs be co-located in the same computer?).

6.4 A joint federal-provincial committee should coordinate the development of models by various provincial water resource agencies. It is difficult for AES to develop a single format for data inputs to hydrological models if there are a number of models being used in different parts of the country and each one requires different inputs.

6.5 A joint review of major operational hydrological models should be conducted by IWD and AES scientists. This review should identify those areas for improvement in the models that involve the way in which the models use meteorological inputs. Particular attention should be given to the points raised in Section 5.

6.6 When launching a new project requiring significant meteorological input, regional IWD staff and provincial hydrometeorological Sections should be encouraged to contact their AES Regional Headquarters. A list of contacts is given in Appendix A. This procedure will ensure that they gain access to the best regional and headquarter's expertise.

6.7 In addition to the sensitivity analyses recommended in 6.1, IWD should document the requirements of hydrological models for short-term and long-term forecast information. This would include statements regarding the times of issue, the areas for which forecast information is required, the time intervals for which precipitation amounts are required, the most suitable formats for presenting the information, the level of accuracy required, the requirement for updates, the thresholds of precipitation amounts for probability of precipitation forecasts on both a daily and a monthly basis, etc.

7. Acknowledgements

I wish to express my appreciation to Messrs. W. Pugsley, P. Louie and L. Mapanao of the Canadian Climate Centre's Hydrometeorology Division for their informative discussions regarding hydrological models and special meteorological studies and services that have been provided in support of these models.

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Appendix A

Listing of the addresses of regional and headquarters contacts for hydrometeorological problems. Users with regional problems (on a scale of the size of a province or some smaller area) should contact the regional office first.

REGIONAL OFFICES:

Director, Pacific Region
Atmospheric Environment Service
216 - 739 West Hastings Street
VANCOUVER, B.C. V6C 1A1
Attn: Scientific Services Division

Director, Central Region
Atmospheric Environment Service
266 Graham Avenue, Room 1000
WINNIPEG, Manitoba R3C 3V4
Attn: Scientific Services Division

Director, Quebec Region
Atmospheric Environment Service
100 Alexis Nihon Blvd., 3rd Floor
Ville St-Laurent, P.Q. H4M 2N6
Attn: Scientific Services Division

Director, Western Region
Atmospheric Environment Service
Argyll Centre
6325 - 103 Street
EDMONTON, Alberta T6H 5H6
Attn: Scientific Services Division

Director, Ontario Region
Atmospheric Environment Service
25 St. Clair Ave. East, 3rd Floor
TORONTO, Ontario M4T 1M2
Attn: Scientific Services Division

Director, Atlantic Region
Atmospheric Environment Service
P.O. Box 5000
BEDFORD, N.S. B0N 1B0
Attn: Scientific Services Division

NATIONAL HEADQUARTERS:

Assistant Deputy Minister
Atmospheric Environment
Service
4905 Dufferin Street
Downsview, Ontario
M3H 5T4
Attn: Hydrometeorology
Division

THE NEED FOR A REGIONAL UNIT HYDROGRAPH ANALYSIS OF ONTARIO

Dale Kimmett
Water Survey of Canada
Ontario Region
Guelph, Ontario

Technical Workshop
"Modelling Activities Related To Flood-Damage Reduction"
Place Vincent Massey, Hull, P. Q.
May 23-25, 1979

Sponsored By
Department of Environment

ABSTRACT

The Canada-Ontario Flood Damage Reduction and Hydrometric Cost-Share Agreements have stimulated an increased interest and action in provincial water resource management. Historically the Conservation Authorities and the over forty consulting engineering firms have had a major vested interest in water resource problems in general and flood damage reduction in particular.

Paralleling this renewed activity has been the upward demand for short time interval streamflow data from the Ontario Region office of Water Survey of Canada in Guelph. Most data of this type is being used for calibrating and sensitivity analysis of the U.S. Soil Conservation and Agriculture Research Service unit hydrograph models known as S.C.S. and HYMO respectively.

Frequent personal contact and telephone discussions with these data users revealed that there is consensus that many of the credibility problems associated with past hydrologic studies are related to the fact that each individual has a different understanding of unit hydrograph theory and how it is best applied. It is proposed that Water Survey of Canada develop the necessary in-house expertise and publish unit hydrograph(s) on a continuing as required basis for appropriate hydrometric stations. Recent technological advancements, the need for standards in hydrology generally, and the availability of sufficient data are further incentives for initiating such a program at this time. An informal canvass of several data users drew favourable responses to the concept. A conceptual look at what should be published is provided.

Introduction

Historically, the majority of hydrological studies and modelling has been concerned with the solution of specific engineering problems such as design of water resource structures, site flood plain mapping and water supply to name a few. Earlier techniques involved synthetic unit hydrograph and rational methods and to some extent unit hydrograph models employing daily data when available.

In more recent years, as water resource modelling has become more prevalent in Ontario, a parallel increase in the demand for short time interval data has been observed. For example, in 1977 alone, fifty out of a total of four hundred individual requests were for hourly and/or six hourly data. These requests involve almost all of southwestern Ontario as can be observed by referring to Figure 1 displaying the stations involved. The fulfilling of these requests at present is a time consuming and somewhat costly process since such data must be generated from the digitized record for each request. In view of this, persons requesting data are quiried on their specific needs, uses of data and other factors that may reduce the resources required in fulfilling these requests.

Through this frequent dialogue with the users, who include almost everyone involved with water resources in Ontario, several interesting facts, common problems, and trends have been observed concerning past, present and planned water resource studies in Ontario.

The majority of studies, past and present, carried out by Conservation Authorities were contracted out to private consulting engineering companies. Recently, the availability of federal funds under the FDR Cost-Share Agreement has stimulated a review and scrutiny of previous flood risk measures and maps (in addition to new studies) to determine if they comply with federal specifications.

This re-assessment by Authority personnel has brought to light the fact that those portions of the past work that does not comply is due to lack of standards, detailed contract specifications and in particular misuse of unit hydrograph theory. Therefore significant benefits of economy, efficiency and effectiveness should be realized by all users through publication of unit hydrographs at appropriate Water Survey of Canada stations.

2. Major Interested Parties

2.1 General

Rapid industrial development and urbanization throughout Ontario over the last thirty years has necessitated the creation, restructuring and renaming of several provincial government ministries. Many of these agencies such as Ministry of Natural Resources, Ministry of Transportation and Communications, Ministry of Environment, Ministry of Agriculture and Food and others have responsibilities and/or mandates relating to water resource management. Other agencies, such as the Conservation Authorities and Ontario Hydro existing under special ligislation, play significant roles in the water resources field throughout the province. The Federal government through arrangements like the Federal-Provincial Hydrometric and FDR Agreements and other federal mandates also have a vested interest. Superimposed upon all this government involvement and adding to the usual difficulties of obtaining co-ordination and maintaining standards, is the significant role played by the consultant engineering firms. With this in mind a brief review of the responsibilities and activities of some of the major data users in Ontario is provided in the following sections.

2.2 Conservation Authorities Branch and Consultants

A Conservation Authority is an organization established under the Provincial Conservation Authorities Act of 1946. Its unit of operation is the watershed and the Province of Ontario gives financial and technical assistance to the Authorities through Conservation Authorities Branch headquarters in Toronto and the eight regional offices of the Ontario Ministry of Natural Resources. Although the Conservation Authorities are legal entities, effective control of all Authority engineering work (studies and construction) is maintained by the Ministry through the provision of fifty-five percent of the necessary funds.

Of the thirty-eight existing Authorities (Figure 1) several have carried out studies related to specific flooding problems. However, only recently, have comprehensive maximum flood studies been initiated.

The majority of these studies were performed by consultants (Figure 1) according to a Terms of Reference supplied by the Authority. Some pertinent excerpts respecting the hydrology requirements of the Terms of Reference are as follows:

1. Flow must be generated for the Regional Storm (or event as per Figure 2) as well as the 100, 50, 25, 10 and 2 year storms.

2. The unrouted flows must be generated using the United States Soil Conservation Service Method. The S.C.S. antecedant conditions are also specified for the events.

3. Other models such as HYMO may be used subject to approval of the Regional Engineer of the Ontario Ministry of Natural Resources.

Past practice of consultants has been to use rational or unit hydrograph methods to generate flows, since the Regional Flood criteria for most of Ontario is the flow from a design rainfall event. In cases where a frequency analysis method might be employed there is seldom sufficient streamflow record available. Until recently the United States Soil Conservation Service S.C.S. model was a favourite tool of the consultants, however the HYMO model is now the most popular model.

The Hydrometeorological Unit of the Conservation Authorities Branch in Toronto as well as the local Authorities have mandates for providing flood advisories and warnings in Ontario. Real-time meteorological and water level data is necessary for immediate interpretation and predicting of the likely resulting runoff. This runoff is estimated using stage-discharge tables and previously developed unit hydrographs.

2.3 Ontario Ministry of Transportation and Communications

The ministry has estimated that it has transportation systems involving 12,000 major river crossings and up to 500,000 minor river crossings with a minor crossing defined as having an overall project cost up to ten thousand dollars.

Historically most of design flows associated with bridges, culverts, drainage etc. have been derived by in-house personnel with perhaps 10-20% of the work now being contracted out to consulting firms and a probable increase in the use of consultants in the future.

Four methods have been and are still being used in obtaining design return flows of frequency from 2 to 100 years. The methods are as follows:

1. Frequency Analysis when sufficient record is available.
2. Watershed Classification Method (developed in-house).
3. Unit Hydrograph Methods.
4. S.C.S. and/or HYMO calibrated models.

Present contracts being let by the Ministry recommend the use of models such as S.C.S. or HYMO. An evaluation of HYMO is now underway at Ministry of Transportations and Communications Headquarters in Toronto.

The Ministry is now preparing a users manual recommending the use of specific methods for particular problems within the Ministry.

2.4 HYMO and S.C.S. Models

The unit hydrograph model known as S.C.S. was developed in the late 1960's by the United States Soil Conservation Service for predicting runoff from ungauged basins. As observed in Figure 3 the unit hydrograph is based on a triangular configuration and assumes that this shape is a function only of time to peak.

In addition to its procedure for estimating runoff volume from rainfall, the Soil Conservation Service has proposed empirical equations for synthesizing unit hydrographs based on studies of many watersheds in the United States.

The unit hydrograph model known as HYMO was developed in the early 1970's by the U. S. Agricultural Research Service. HYMO was initally used in Texas, later in the Western States and has now been adopted by many engineers in Ontario. The unit hydrograph employed by HYMO as seen in Figure 3 is dimensionless and is defined by three distinct equations. One equation defines the co-ordinates from the point of rise to the inflection point while the remaining two define the recession limb. The HYMO model is now being accepted as a better model than the S.C.S. by the majority of users in Ontario.

3. Problems Related to Water Resources in Ontario

3.1 General

Many of the problems in Ontario are related to the fact that historically agencies involved in water resource programs have carried out their responsibilities in relative isolation to one another. However, the Federal-Provincial Hydrometric and Flood Damage Reduction Cost-Sharing Agreements are two examples where the effect has been to force a more co-ordinated approach to improving water management practices in Ontario.

This closer contact with the provincial personnel in addition to the almost continuous contact with the consultants, academics and other data users has brought to light some of the observations expressed in the following section.

3.2 Common Observations Expressed by Data Users

1. Almost all consultants are using the unit hydrograph model HYMO but insufficient care is being taken in the event selection prior to calibration.
2. "Terms of Reference" given to consultants are not specific enough.
3. There has been little if any co-ordination between one Authority and another.
4. In some cases the practice of selecting the lowest bid has led to engineering "shortcuts".
5. The consultants' final reports are not sufficiently comprehensive with respect to data used, techniques employed and assumptions made.
6. Unit hydrograph methods are being used inappropriately (eg. for snowmelt runoff).
7. The non-stationarity of streamflow record is often not accounted for when deriving and/or applying unit hydrographs.
8. The use of "canned" models using unit hydrograph methods requires special skill because of the importance of antecedent conditions and the applicability of the unit hydrograph derived.

9. Occasionally two distinct studies will be carried out in the same watershed at or near the same time and different flows for the same event frequency may be derived and used.

10. More than one unit hydrograph is often necessary due to the significant non-linearity of the watershed response.

11. Unit hydrograph methods are being used for analysis of snowmelt events because there is no better method available within the given time and money constraints.

4. Proposal For A Regional Unit Hydrograph Analysis

4.1 Proposal Contents

In view of the fact that such a large proportion of the water resource engineering community utilizes unit hydrograph methods and/or models and since the observations previously discussed hinge mostly on problems with unit hydrographs, a proposal for a regional unit hydrograph analysis was conceived as follows:

1. Unit Hydrographs be developed for appropriate watersheds where sufficient quality and quantity of streamflow and rainfall record exist.

2. The antecedent and event meteorological and runoff data used, as well as assumptions made, be documented and published along with the IUH.

3. Water Survey of Canada develop original and updated unit hydrographs and publish them as a new component of the Surface Water Data series.

4. A committee be established to investigate the feasibility of implementing such a project within the Water Survey of Canada programs especially in Ontario Region.

4.2 Users Reaction to Proposal

Informal canvassing of some of the major consultants, academics and provincial government engineers familiar with unit hydrograph techniques were asked their opinion of the above proposal. Some of those responses are provided in the following sections.

4.2.1 Consultants' Comments

"Since often a number of different studies may be carried out on the same watershed or in the same area, the existence of a Water Survey of Canada unit hydrograph would lead to an improvement in standardization and an economy of effort."

"Many of the computerized models now utilized by many consultants (e.g. HYMO, STORM, etc.) require the input, either directly or indirectly, of a unit hydrograph. If a unit hydrograph were available at a Water Survey of Canada gauge on the basin of interest, this could be used either as input to the model or as a basis for calibrating the parameters of the model's own unit hydrograph. If a gauge were not available on the basin of interest, then the model might be calibrated using unit hydrographs from nearby streams of a similar type. Based on this exercise, a set of parameters for the basin of interest could be developed."

"It might be advantageous to develop the unit hydrographs in a dimensionless instantaneous form, i.e. Q/Q_p versus t/t_p where Q_p is the peak flow and tp the time to peak of the unit hydrograph. In this way, an equation of tp versus intensity of event could be developed if desired and applied to the dimensionless UH."

"Finally, as some gauges may be affected by urbanization which has taken place during their period of record, this should be clearly identified and the conditions to which the UH apply should be stated."

"A number of investigators have suggested that the shape of the unit hydrograph changes as the severity of the event increases. Many have suggested that the time to peak reduces as the event becomes more severe. Some examination of this aspect of the unit hydrograph would perhaps be necessary."

"With regard to mapping alone, we have carried out somewhere between 350 to 400 miles of mapping on various rivers. Assuming an average flood plain width of 600 feet this would correspond to approximately 27,000 acres of flood plains. Needless to say, predefined analyses of unit hydrographs would save us $1,000's in preliminary engineering work and I feel would significantly reduce total construction costs. I thus would be more than pleased to support, in any way I am able, your proposal."

4.2.2 Conservation Authority Engineers' Comments

"The unit hydrograph approach to the time distribution of runoff appears to be the most universally used technique and will probably continue to be in the foreseeable future. Compared to other available techniques, it is simple to use and has been shown to give adequate results. Concern has been expressed about the variability of unit hydrographs with runoff magnitude. The user would be able to judge the validity for his purposes of using the published unit hydrograph if the date and runoff volume of the events used to derive the unit hydrograph are specified."

"It is my feeling that the dependence on empirical equations for the development of unit hydrographs on gauged rivers is purely a matter of convenience. Because many hydrographs have to be assessed in order to find a few worthy of hydrograph analysis, time constraints do not always permit an adequate analysis of observed data. In addition, short interval streamflow data is not published and one hesitates to make the extremely large requests for data necessary for unit hydrograph derivation, especially since most of it, on inspection, is not applicable."

"The Grand River Conservation Authority operates a real-time flood forecasting model developed to assist with the operation of reservoirs and to provide warning of impending flood levels. Unit hydrographs are used to predict these flows."

"Under the Grand River Basin Water Management Study, we plan to use unit hydrographs to simulate flood flows of lower magnitudes than the regional flood (25 to 200 year return period). These flows will be used to study the effect of various structural flood control alternatives on flood damages."

"The advantages of your proposal to derive unit hydrographs for selected basins are numerous. I have summarized below a few which I feel are particularly important:

1. Because short interval data is housed with Water Survey of Canada, it is logical that analysis could most efficiently be accomplished there. It is possible for Water Survey of Canada to make the most complete analysis as all available data can be assessed.
2. The duplication of effort of hydrologists would be reduced by the publication of region-wide analysis of river response.
3. A regional analysis would permit Water Survey of Canada to evaluate the applicability of the empirical equations for use on ungauged basins in Ontario/Canada.
4. Transfer of unit hydrograph information between similar gauged and ungauged basins (a recognized, though difficult technique) could be more easily accomplished.
5. Such concerns as the variability of the unit hydrograph with magnitude could be assessed where a complete analysis of many basins is undertaken by one agency."

"Since our prime model is "HYMO" which uses unit hydrograph information, your proposal to prepare Unit Hydrographs for our streams is very timely. I can also see the possibility of utilizing Unit Hydrographs on our "Flood Warning" function, to give us a predictive tool for flood flows."

"Finally, I can see the preparation of a Unit Hydrograph for our area as being very helpful to other consultants who are required to provide us with flood flows and elevations in order to comply with our Fill and Flood Line Regulations. A uniform hydrograph would make calculations and administration easier by having all calculations conform to a standard."

4.2.3 Other Comments

"Since watersheds are dynamic (change with time) there will be a need for a continuing monitoring of the watershed response and updating of published unit hydrographs as required. Water Survey of Canada staff spend a great deal of time on the watershed and handling the raw data and thus they would be best suited for identifying the need for and updating of these unit hydrographs."

5. Why Do Unit Hydrograph Analysis Now?

There have been some recent developments that would enhance the undertaking of such a study at this time. First and foremost there are now over 350 streamflow gauging stations in Ontario having ten or more years of continuous record. This relative abundance of data (mostly in Southern Ontario) is further complemented by the fact that approximately 250 of these same stations have digitized continuous record making it feasible for automated generation of short time interval data (i.e. hourly data). Continuous record prior to 1969 must be inspected and data extracted manually from original records housed by Water Survey of Canada in Guelph.

A second noteworthy development is recent work carried out by the Atmospheric Environment Service at the Canadian Climate Centre in Toronto. The Hydrometeorology Division has developed an automated procedure for scanning their one hour historical tipping-bucket rain gauge data in order to select and analyze specific rainfall events. Twelve meteorological stations in Ontario have been analyzed for the maximum one hour intensity event. The data required for one hour events is still abstracted manually, however the format as shown in Figure 4 will be maintained for the automated abstraction and analysis of the maximum 6 and 12 hour events now underway. These analyses used in conjunction with the short time interval discharge data available from Water Survey of Canada should reduce the time consuming task of meteorological data acquisition, event selection and data preparation.

Finally as previously pointed out it appears that the majority of modellers are and will be using a common unit hydrograph model namely, HYMO. This general acceptance of a common tool especially by the consulting companies is a step in the right direction and should be complemented with a "standard" unit hydrograph. Another development in the same vein is the recent development by the Conservation Authorities Branch Headquarters of a model named HYDRO1. This unit hydrograph model allows for taking into account the non-linear response and is planned to be made available to the Authorities and the consultants in the near future. The University of Guelph will be preparing a "Methodology for Calibration" of the HYDRO1 model.

Where feasible then, unit hydrographs derived from observed data available in published form would be used in preference to synthesizing unit hydrographs or employing empirical formulae.

6. What Would Be Published?

Although the proposal puts emphasis on the publishing of the unit hydrograph(s) on a regional basis it is also at least as important to document the raw data, assumptions and methods used. The items considered necessary to be published are as follows:

1. Antecedent rainfall hyetograph and/or table.
2. Storm rainfall hyetograph and/or table.
3. Observed event runoff hydrograph and/or table.
4. Baseflow flow rates for the event.
5. Watershed physiographic characteristics.

6. Map indicating location of rainfall and streamflow gauging stations.
7. Length and duration of available record.
8. Unit Hydrograph(s) for the event(s).

The U.S. Department of Agruculture have published similar information to that covered in items one through seven in their Miscellaneous Publication No. 994 entitled "Hydrological Data For Experimental Agriculture Watersheds In The United States 1960-1961".

7. Summary and Conclusions

In summary the use of unit hydrograph theory, has been, is and will for the foreseeable future continue to be the mainstay of the working hydrologist. This is not only due to its being a basic concept of watershed response but also it is an efficient and relatively simple, reliable tool in the study of rainfall-runoff transformation when employed with care.

Historically much of the work has been carried out in piecemeal fashion to satisfy a particular need at the time. Tools available to the hydrologist for calculating streamflow consisted of manual methods such as the rational method, unit hydrograph techniques as well as the recent use of unit hydrograph models such as S.C.S. and HYMO. Hydrological models are a fairly recent innovation in terms of being an acceptable tool useable by the majority of the hydrological community.

In addition to lack of technical tools varied degrees of in-house expertise, lack of standards, insufficient contract specifications, political makeup of the Conservation Authorities, selection of the lowest bidder and inexperienced management have contributed in many cases to a lack of credibility and suspicion of work carried out in the past.

It appears that the HYMO model is emerging as the best available hydrologic tool (within economic and time restraints) not only for the consulting engineering firms but also for many provincial government agencies having available in-house expertise.

The publishing of initial and undated versions of unit hydrographs by Water Survey of Canada would provide all the benefits normally attributed to standardization and provide many spinoff benefits if implemented.

CONSERVATION AUTHORITIES IN ONTARIO

Ausable Bayfield
Cataraqui Region
Catfish Creek
Central Lake Ontario
Credit Valley
Crowe Valley
Essex Region
Ganaraska Region
Grand River
Halton Region
Hamilton Region
Kettle Creek
Lakehead Region
Long Point Region
Lower Thames Valley
Lower Trent Region
Maitland Valley
Mattagami Region
Metropolitan Toronto and Region
Mississippi Valley
Moira River
Napanee Region
Niagara Peninsula
Nickel District
North Bay - Mattawa
North Grey Region
Nottawasaga Valley
Otonabee Region
Prince Edward Region
Raisin Region
Rideau Valley
Sauble Valley
Saugeen Valley
Sault Ste. Marie Region
South Lake Simcoe
South Nation River
St. Clair Region
Upper Thames River

CONSULTING ENGINEERING COMPANIES IN ONTARIO INVOLVED WITH WATER RESOURCES

Acres International Limited
Ainley and Associates Ltd.
Airphoto Analysis & Associates
R. B. Anderson Assoc. Ltd.
Beak Consultants Limited
Bird and Hale Limited
E. A. Bishop Associates Ltd.
Bunting Wardman Limited
R. J. Burnside & Associates
Chrysler & Lathem Limited
Dames - Moore Consultants
Dick Consulting Service
M. M. Dillon Limited

Ecoplans Limited
Foster Engineering Limited
Gamsby and Manerow Consulting
Gartner Lee Associates
Geocan Consulting Engineers
Golder and Associates
Gore and Storrie Limited
Greer, Galloway & Associates
Gregg Edens Limited
Hunter and Associates
Indecon Consultants Limited
Kilborn Engineering Limited
W. P. London & Assoc. Ltd.
James F. MacLaren Limited
Marshall, Macklin & Monaghan Consulting
Robert McAlpine Limited
McCalla and Associates
McCormick, Rankin & Associates Ltd.
Northland Engineering
C. C. Parker and Associates
Phillips Planning & Engineering
Polutech Limited
Proctor and Redfern Limited
J.L.Richards & Associates Ltd.
A.J.Robinson & Associates Ltd.
B. M. Ross & Associates
Shawinigan Engineering Limited
Simcoe Engineering Limited
Springbank Consulting Engineers Ltd.
Triton Engineering Services Ltd.
Underwood, McLellan Limited
Walter, Fedy, McCargar & Hachburn

FIGURE 1

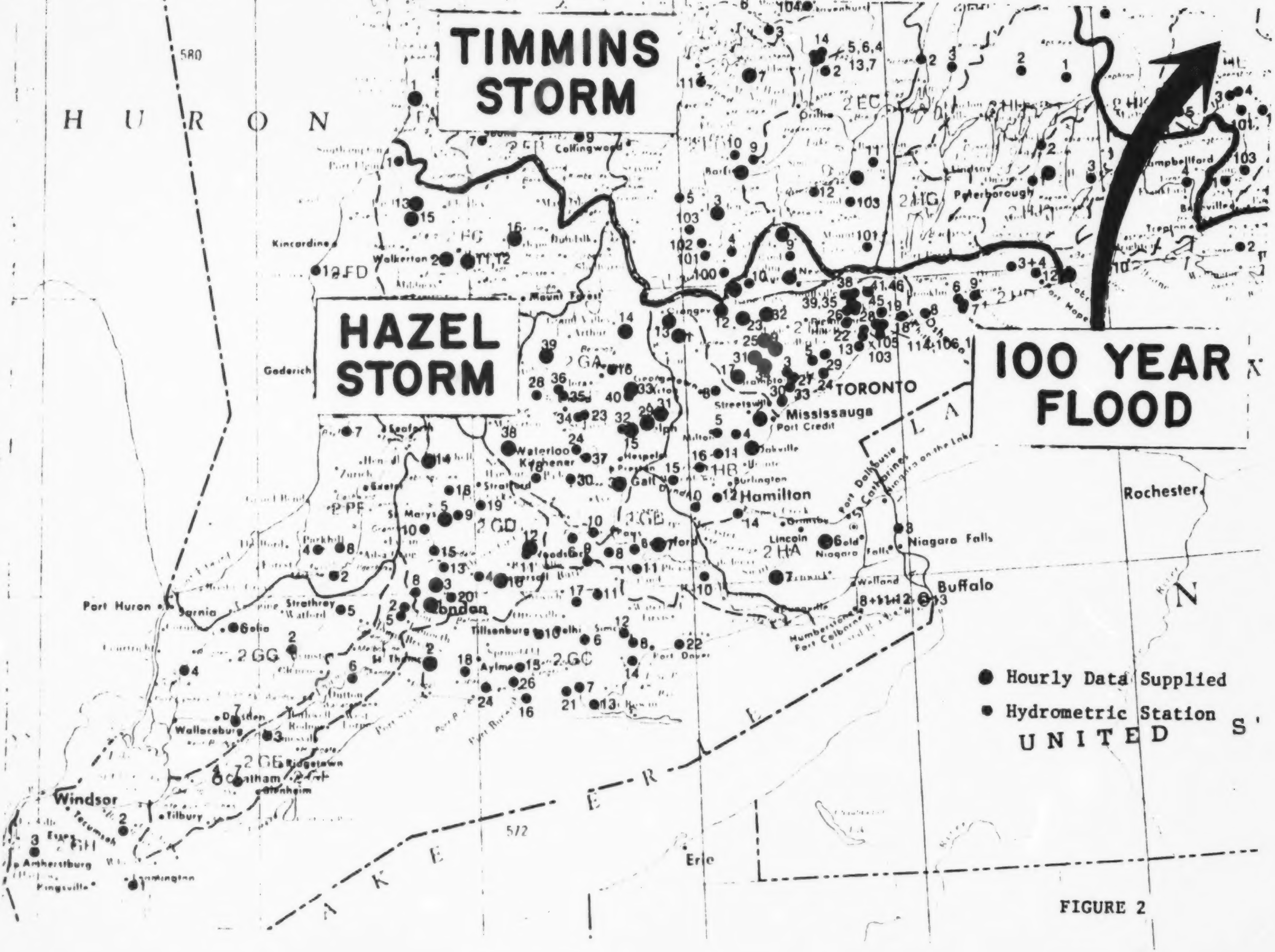
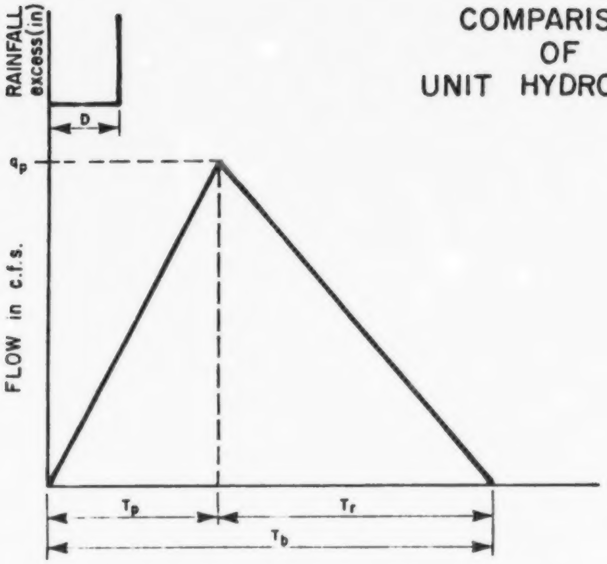
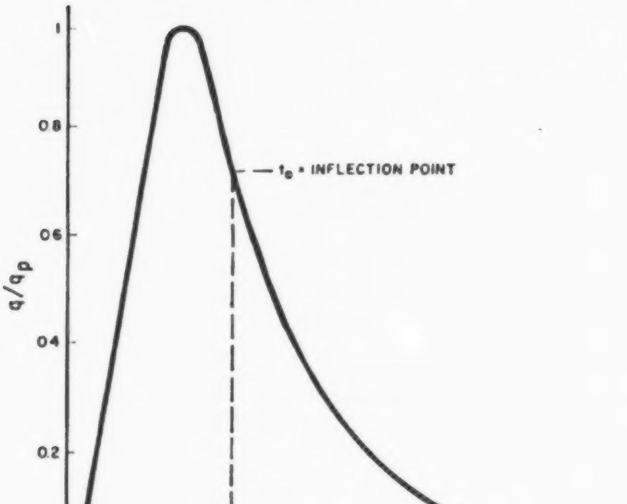

TIMMINS STORM
HAZEL STORM
100 YEAR FLOOD
● Hourly Data Supplied
• Hydrometric Station
H U R O N
L A K E   E R I E
UNITED STATES
TORONTO
Mississauga
Port Credit
Hamilton
Burlington
Oakville
Niagara Falls
Buffalo
Rochester
Welland
Port Colborne
Collingwood
Barrie
Peterborough
Lindsay
Campbellford
Belleville
Port Hope
Oshawa
Kincardine
Walkerton
Goderich
Mount Forest
Seaforth
Exeter
St Marys
Stratford
Waterloo
Kitchener
Guelph
Galt
Streetsville
Brampton
Orangeville
Woodstock
Brantford
London
St Thomas
Aylmer
Tillsonburg
Simcoe
Delhi
Port Dover
Strathroy
Sarnia
Port Huron
Wallaceburg
Dresden
Chatham
Ridgetown
Blenheim
Tilbury
Windsor
Tecumseh
Essex
Amherstburg
Kingsville
Leamington
Erie
Lincoln
Grimsby
Port Dalhousie
St Catharines
Niagara on the Lake
Humberstone
2 EC
2 HG
2 HB
2 HA
2 GA
2 GB
2 GD
2 GC
2 GG
2 GE
2 GH
2 FD
2 FC
2 FE
2 HJ
2 HK
580
572

FIGURE 2

COMPARISON
OF
UNIT HYDROGRAPHS


RAINFALL excess(in)
D
q_p
FLOW in c.f.s.
T_p
T_r
T_b

SOILS CONSERVATION SERVICE
UNIT HYDROGRAPH



DIMENSIONLESS UNIT
HYDROGRAPH USED IN HYMO

FIGURE 3

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              ONE HOUR RAIN EVENTS AND THEIR 5-DAY ANTECEDENT RAINFALL
                  (RAINFALL AMOUNTS IN MILLIMETRES X 10)


STATION:     OTTAWA ONTARIO

SELECTION CRITERIA(MM*10):5 MIN    1 HR
                           *       178

MONTH                          I     II    III    IV     V    VI    VII  VIII    IX     X    XI   XII
MEAN MONTHLY RAIN            160    130    274   607   686   742   810   864   818   655   594   284
MEAN #DAYS WITH NO RAIN       28     26     26    20    20    19    20    21    19    20    20    26

 EVENT DATE ANTECEDENT RAINFALL           MAX  MAX
 YR MO DA   DAY 0  -1  -2  -3  -4  -5     5 M  HR DAY      5 MIN VALUES IN MAX HR,UNCORRECTED        MAX SUM
 60  5 31     376  28   0   0   0   0      61 196  91      3  3 13 15  3 23 51 36 41 20  5  3         51 216
 61  7 21     251   0   0   0   0  20     107 249   3      8 25  0  5 10 25 33102 10  8  5  3        102 234
 62  7 12     317  20   0  20   0   0      61 290   0      0  5 20 25 41 56 46 46 25 13  3  0         56 280
 62  8 27     185   0   0   0   0   0      33 180   3      0  5 15  5 33 25 13 28 30 20  0  0         38 174
 63  5 14     206   5   0   0   0   0     104 180   0      0  0  0  0 58 84 15  0  0  0  0  0         84 157
 64  7 12     305 165   3   0   0   0      61 236  48     25 36 41 41 25 15 10  8  5  3  5  3         41 217
 64  8 11     221   0   0   0   0   0      48 180   0     18 20 13  8  0  5  3  5 23 46 36 13         46 190
 65  8  7     246 315   0   0  23 173      61 183  33     46 41 33  3  3  3 10  8 13  3  3  5         46 171
 65  9 18     183   0   0 170   0   0      81 180   3     28 81 41 10  0  0  0  0  0  5  8  3         81 176
 66  8 16     287   0   0   0   5 152      74 231  33      3  3 79 61 30 15 30  3  3  0 10 25         79 262
 67  9 21     475  58   0   0   0   0      56 218 140     51 41 15 10 13 20 18 13 13  8 13 25         51 240
 69  8 18     422  18   8   0   0   0      84 386  36      0  3 48 61 56 97 66 41 36 10  0  0         97 418
 70  7 20     442  53   0   3 178  28     102 231  84     10 41  5 15 20  3  0  3  3 89 18  8         89 215
 70  9 26     208  56  94   0  94  18      71 201   0      8  8  5  5 15 30 56 13 15 15  8  3         56 181
 71  8 14     317  13   0  23  66   0      36 198   8     15 20 41 20 20 10 18  3  5  5  3  3         41 163
 72  7 11     330   0 229   0   0   0     117 330   0      0 15 13 15 20 10 80 15 36107 18  0        107 279
 72  8  7     396   8   0   0 224 122      41 249  91     23 15 15  5 15 30 25 25 10 30  5 20         30 218
 73  6 11     427  36  20  53  81   0      81 414   3     18 41 15 25 41 56 38  0 15 89 15  8         89 361
 73  8  6     236   5   0   0  58 102     109 180  23      0 38 91 28  0  0  0  0  0  3  0  0         91 160
 73  8  8     605  10 236   5   0   0      56 307 211      5  3  8 10 51 41 56 15 30 46  8  0         56 273
 74  5 22     312   0   0   0   0   3     112 216   5      0  0  0 13  5  3  5  5 15 28104  0        104 178
                                                    ↑
                                                    └RAINFALL ON DAY PRECEDING HOUR
                                                     OF MAXIMUM RAINFALL
MEAN STORM DISTRIBUTION          5 MIN VALUES (%) FROM STORMS CENTRED ON INTERVAL 13
                          1  2  3  4  5  6  7  8  9 10 11 12MAX 14 15 16 17 18 19 20 21 22 23 24
                          0  0  0  1  2  1  2  2  2  4  5 12 32 13  8  5  4  2  1  1  1  1  0  1
```

FIGURE 4

WIXON CREEK WATERSHED

EVENT COMMENCING 750418 at 20:00 Hr.

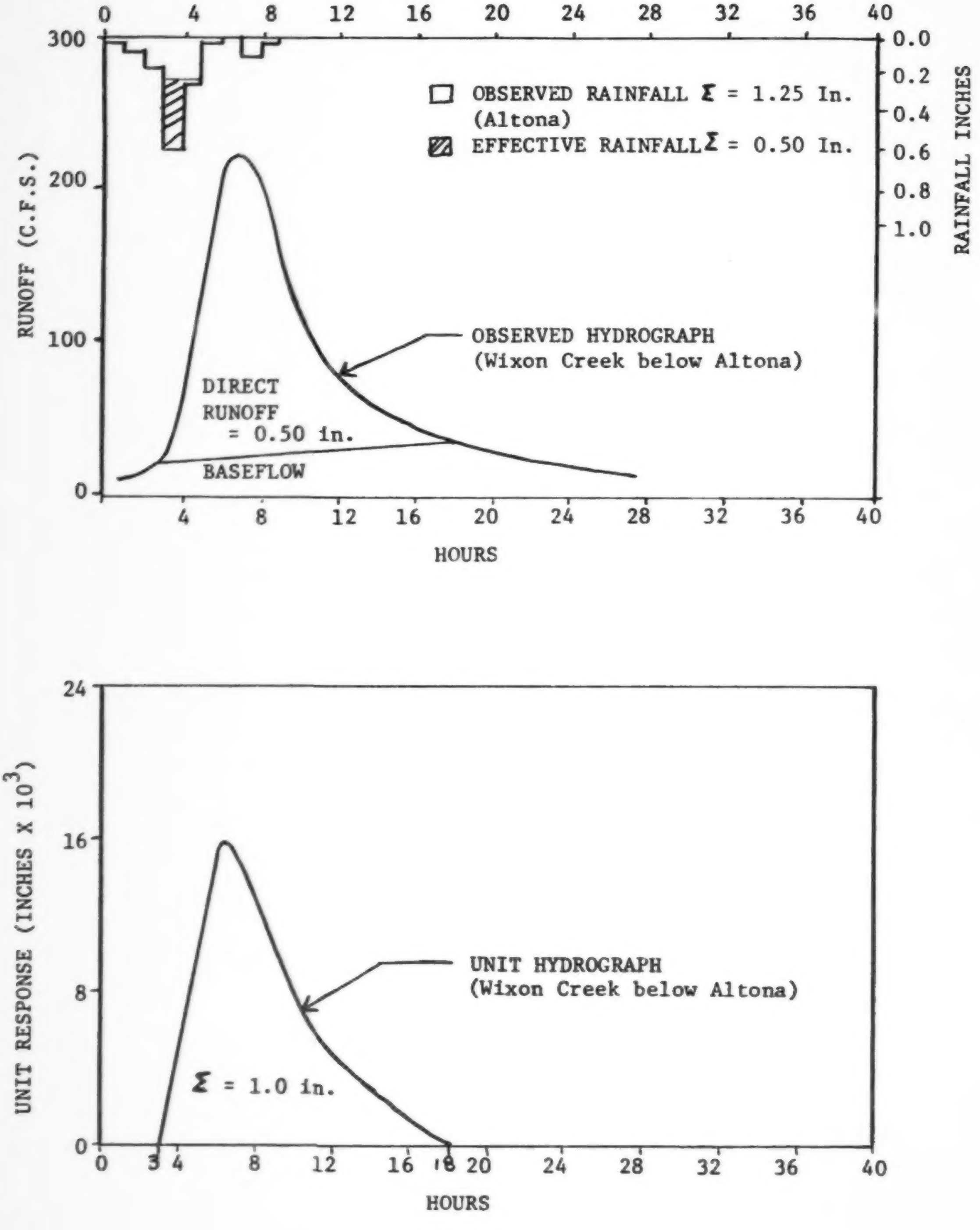

FIGURE 5

Submitted to Technical Workshop - "Modelling Activities Related to Flood Damage Reduction" to be presented on May 23-25, 1979 in Ottawa

RUNOFF FROM DEVELOPING AREAS

by

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April 1979

RUNOFF FROM DEVELOPING AREAS

J. Marsalek, Hydraulics Research Division
National Water Research Institute, Burlington, Ontario.

1.0 INTRODUCTION

Population growth and individual mobility have resulted in a continued encroachment of rural land by residential, commerical, and industrial development in Canada. The process of land development which is often referred to as urbanization was particularly fast during the past 30 years when the urban population of Canada increased from just over one-half to over three-quarters of the total population. If these past trends continue, the urbanization process and population growth will, by the end of this century, increase the urban population of Canada by an additional 10 million people and further urban growth and land development can be expected.

Urbanization alters the hydrologic response of a watershed because of attendant increases in the watershed imperviousness, and in runoff removal efficiency. This alteration then leads to an increase in surface runoff with attendant increases in the occurrence of flooding and degradation of the quality of surface runoff. A number of studies have attempted to measure the changes in the watershed runoff response caused by urbanization. A recent summary of reported results (3) indicates that urbanization may increase the runoff volume up to six times, increase the peak runoff rates up to ten times, and reduce the response time up to ten times. The reported increases in runoff peak rates are plotted in Fig. 1 for a wide range of return periods, watershed sizes, and degrees of development. The data in Fig. 1 substantiate the notion of increasing runoff peak rate due to urbanization and indicate that the effects of urbanization on rare floods (such as the 100-year event) are significantly smaller than on frequent floods (e.g. the mean annual event).

The problem of runoff from developing areas is often addressed by various government agencies in connection with flood protection studies. Hydrologic modelling is the basic tool which is used in such studies. Selected aspects of modelling runoff from developing areas are discussed in the following presentation which attempts to follow the IWD Workshop format rather than to give a general treatment of the discussed subject.

2.0 RUNOFF MODELS

Runoff from developing areas is studied in urban hydrology which is recognized as a distinct field of hydrology with its own set of predictive models and expressions for representing the hydrologic response of developing watersheds. A brief description of rainfall-runoff processes in urbanizing watersheds and overview of selected runoff models are given below.

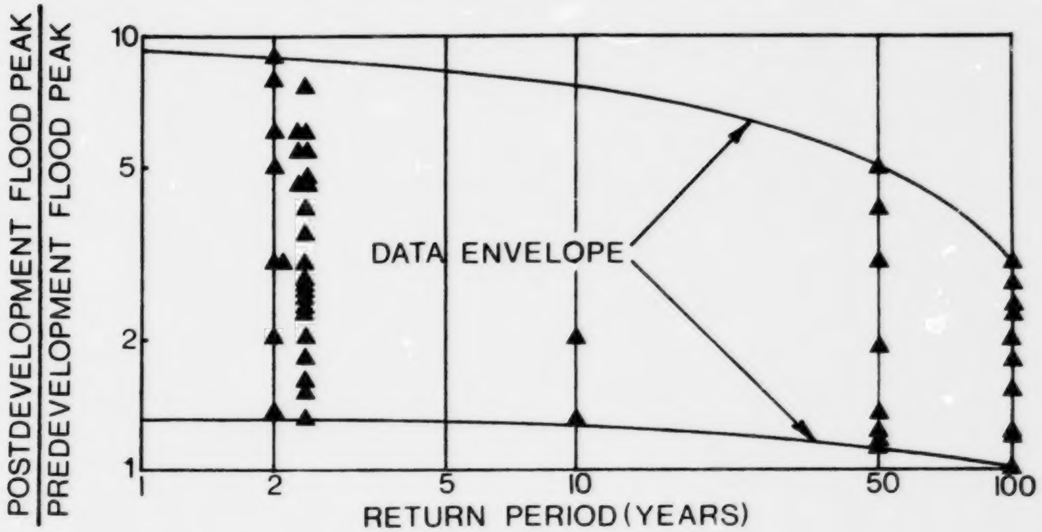

FIG. 1. EFFECTS OF WATERSHED DEVELOPMENT ON FLOOD PEAKS (DATA AFTER REF. 3)

2.1 Rainfall-Runoff Process

The rainfall-runoff process on a developing watershed consists of hydrologic and hydraulic components. Rainfall, rainfall losses and the resulting overland flow comprise the hydrologic component which produces an inlet hydrograph. The summation of inlet hydrographs and the routing of the resultant flow through a transport network comprise the hydraulic component.

The runoff volume is determined from a known rainfall by estimating the rainfall losses and their variation in time. For calculation of rainfall losses, impervious and pervious surfaces are treated separately. On impervious surfaces, a zero infiltration is assumed and the only loss considered is the surface depression storage which is typically assigned a constant depth over the entire area. Runoff from impervious areas can start even before the surface depression storage is completely filled. On pervious areas, rainfall water is continuously being abstracted by infiltration. Other losses, such as interception and evapotranspiration, are usually neglected. Once the rainfall exceeds the infiltration rates, the excess flow starts to fill the surface depression storage. While some runoff may occur at this stage, the entire area starts to contribute to the total runoff only after all the depressions have been filled.

Surface runoff from both pervious and impervious areas forms an overland flow and is transported towards drainage ditches and gutters and thereby to drainage pipes or channels. At inlet points, various contributions are synthesized into inlet hydrographs and these are then routed through drainage networks. In general, the hydraulics of flow routing, although complex for looping and converging and diverging branch systems with special structures, is the best understood part of the runoff process.

The rainfall-runoff process is simulated by means of various hydrologic models which are overviewed in the following paragraph.

2.2 Runoff Models for Developing Areas

A runoff model in its elementary form is simply a group of mathematical expressions that simulate the process of the conversion of rainfall to runoff. Runoff models are sometimes classified according to the conceptual approach taken, mode of model operation and the level of modelling.

The conceptual approach to runoff modelling varies from physically based models to black-box models. Experience shows that the boundary between both approaches is rather diffuse and that both approaches can be successfully used in practice.

In another classification, runoff models are classified as discontinuous and continuous. Discontinuous models (or single-event models) simulate the watershed response for a discrete rainfall event. These models which are particularly popular in the urban drainage design do not include any continuous moisture accounting process. Continuous models make a continuous accounting of the movement of all water through the various watershed stores.

Finally, the urban runoff models can be classified, according to the level of modelling, as planning, design, and operational models.

Planning models give an overall assessment of the urban runoff problem and may also provide estimates of the effectiveness and costs of alternative storm runoff management procedures. Relatively large time steps (hours) and long-term simulation periods (months and years) generally characterize these broad-objective models. Minimum data requirements and low mathematical complexity are typical. Long-term planning models may also generate initial conditions for input to design models. The effects of urbanization are readily computed.

Design models generally involve the simulation of selected storm events with short-time steps such as minutes and short simulation periods (hours). Such models can be used for a complete description of flow, storage and pollution routing from the point of rainfall through the total drainage system and into the receiving waters. As with planning models, design models can be used to arrive at least-cost abatement procedures for both quantity and quality problems. Data requirements can be moderate to very extensive depending on the particular model involved.

Operational models help resolve actual control decisions during a storm event. From telemetered rain and flow gauge signals as inputs to the model, estimated system responses are projected a short time into the future. In-system storage, regulator settings, or diversions, or combinations of these, may then be employed as control options. Informational needs for operations models are much greater than for either planning or design models.

A considerable number of models have been developed in the last few years for the analysis of runoff from developing areas. A detailed description and assessment of these models was given elsewhere (1, 8). Features of selected urban runoff models are listed in Table 1. The data in Table 1 indicate a wide range of features of the existing urban runoff models. Where user's needs cannot be satisfied by a single model, one can interface two or more models to obtain the desired modelling tool.

TABLE 1. FEATURES OF SELECTED URBAN RUNOFF MODELS(AFTER REF.2)

MODEL ORIGIN	MODEL ABBREVIATION	MODEL FEATURES									
		MULTIPLE HYETOGRAPHS	MULTIPLE CATCHMENTS	RUNOFF FROM PERVIOUS AND IMP. AREAS	SNOWMELT	WATER BALANCE BETWEEN STORMS	SEWER FLOW ROUTING	UPSTR. AND DOWNSTR. FLOW CONTROLS	PRESSURE FLOW	STORAGE	PRINTOUT OF STAGE AND VELOCITIES
1 BRITISH ROAD RES. LAB	RRL	●	●				●				
2 CORPS OF ENGINEERS	STORM			●	●	●				●	
3 DORSCH CONSULT	QQS	●	●	●		●	●	●	●	●	●
4 ENV.PROTECTION AGENCY	SWMM	●	●	●	●		●	●	●	●	●
5 HYDROCOMP	HSP	●	●	●	●	●	●	●		●	●
6 ILLINOIS STATE	ILLUDAS	●	●	●			●			●	
7 QUEEN'S UNIVERSITY	QUURM		●	●			●				
8 UNIV. OF CINCINNATI	UCURM		●	●			●				

3.0 INLAND WATERS DIRECTORATE EXPERIENCE

Much of the IWD experience in modelling runoff from developing areas stems from the Research Program on Urban Drainage under the Canada-Ontario Agreement on Great Lakes Water Quality (7). In this program, a need to provide a modelling tool for the assessment of runoff from urban areas was identified. Further study indicated that this need would be best met by a package of models which could serve for both continuous and discontinuous simulation of runoff and would be interfaced with the Canadian data banks.

The interface between the data banks and runoff models was achieved by means of a newly developed Data Analysis Model. This model reads precipitation and temperature data from the data banks, processes these data, and provides data records in formats which meet the requirements of the runoff models included in the package.

For discontinuous detailed simulations of runoff, the Storm Water Management Model of the U.S. Environmental Protection Agency was selected (9, 10). Apart from good reproduction of observed events, the main reasons for selecting SWMM were the availability of a good model documentation and the continued maintenance and refinement of the model by EPA. The SWMM model computes runoff from developing areas, routes the runoff through various transport networks, simulates operation of control schemes for runoff quantity and quality and, finally, simulates the impact of runoff on the receiving waters. For the purpose of the COA program, the SWMM model was modified by extending the model applicability to multi-event simulations, adding a snowmelt subroutine, amending the control schemes, and updating the engineering costs of control schemes.

For continuous simulations of runoff, the STORM model of the U.S. Army was adopted (12). The STORM model computes runoff in hourly steps and simulates the effects of runoff storage and treatment. Snowmelt computations can be included in simulations using the degree-day method. The water balance between the storms is determined via the recovery of depression storage based upon specified evapotranspiration rates. The watershed is considered as a single element and no routing is computed. The main purpose of STORM applications is to obtain a preliminary appreciation of storm-water problems in developing areas. A good understanding of total runoff volumes and required control strategies is obtained at a relatively low cost.

Both models, SWMM and STORM, were verified on a number of Canadian watersheds. The STORM model performed fairly well in identifying frequencies of occurrence of overflow events (11). The SWMM model was subject to detailed verifications which were described elsewhere (5). Very good results were obtained for runoff volumes; the runoff peaks were reproduced by SWMM less accurately. Most of these verifications were done for events with high frequency of occurrence. In such events, the runoff is controlled mostly by contributions from the impervious areas. The lack of data prevented verifications for low frequency events, during which the pervious areas would substantially contribute to the total runoff.

Various models from the package described above have been successfully used in a number of demonstration studies and engineering applications (6). Some features of the modified SWMM model have been subsequently adopted by EPA in their most recent version of the SWMM model.

4.0 DATA NEEDS

Two types of data are required for modelling of runoff from developing areas - model input data and calibration/verification data. Input data for runoff modelling can be broadly classified into three groups - hydrometeorological data, process parameters, and physical (watershed) data.

Depending on the type of a runoff model used, hydrometeorological data required in modelling studies may include precipitation, evaporation, temperature, wind movement, snow depth and solar radiation. In urban applications, the hydrologic synthesis is simplified and, consequently, fewer types of hydrometeorological data are required. Quite often, rainfall data are the only hydrometeorological data used in such studies. The requirements on rainfall data for urban runoff studies were discussed elsewhere (14). For most applications, adequate rainfall data are available from the Atmospheric Environment Service either on magnetic tape files, or through actual daily records on stripcharts.

Hydrologic process parameters quantify the movement and storage of water in and on the land surface of a watershed. Of these parameters, infiltration, surface depression storage, runoff coefficients, and overland flow parameters are particularly important in urban hydrology. Although some process parameters can be measured directly, their measurement may be difficult or impractical and the numerical values of such process parameters are obtained either by calibration or by transposition of data from other watersheds.

The literature data on surface storage, runoff coefficients, and overland flow parameters seem to be sufficient for typical applications (9). There is, however, a definite shortage of soil infiltration data for unsteady rainfalls of high intensity. Such data would be of a particular importance when dealing with urbanizing watersheds of low imperviousness.

The determination of physical parameters of a watershed and of its subareas is a relatively straightforward task. The detail of such data depends on the level of modelling and, under some circumstances, the collection of physical watershed parameters can become a tedious task which significantly contributes to the total cost of modelling. For detailed modelling, the watershed should be characterized as to area; impervious area; effective impervious area; pervious area; contributing pervious area; soil characteristics; inlet characteristics; conduit and open-channel lengths, elevations, slopes, sizes, geometries and friction factors (4). At the planning level, an approximate characterization of the watershed may be acceptable. In fact, the details of the transport networks are frequently disregarded at this level completely.

The second type of data required in runoff modelling are calibration/verification data. The calibration of a runoff model is a procedure in which model parameters are manipulated to reproduce the response of the watershed under study within some range of accuracy. In this procedure, calibration yields estimates of input parameters that are difficult to measure directly, compensates to some extent for imperfections in the model structure, and lends increased reliability to model predictions. Once a model has been calibrated against a set of calibration data, it should be verified with a set of data separate from that used in model calibration. Model verification consists of a rational analysis of both the computed output and any empirically derived parameters. Additionally, to provide a proper verification, the computed model output should be compared with observed output (e.g. runoff flows).

Calibration/verification data for urban runoff models generally consist of fully-documented and synchronized observations of rainfall and runoff for a number of events. The accuracy of observations, and the number and type of calibration/verification events are important.

Direct model calibration (i.e. in the watershed studied) is not always necessary or even possible. Calibrated parameters are often transposed from similar watersheds. Canadian urban runoff calibration/verification data were discussed elsewhere (6). It would appear that there is a small number of urban watersheds from which calibration/verification data could be transposed to other areas. The characteristics of these watersheds and of the observed data yield very little information on the contribution of pervious areas to the total runoff from developing areas.

In addition to runoff modelling data, the cost of flood damages due to flooding of urban areas is important for selecting the design level of flood protection. No such data were reported in the Canadian literature.

5.0 MODELLING PROBLEMS

The modelling problems discussed below are related to model selection, documentations for models, model calibrations and limitations of models.

Because of the large number of models available, it is becoming increasingly confusing for the user to select the model most suited for a particular application. Furthermore, the capabilities of some models were overstated in the literature and the limitations of the models were rarely reported. The lack of information on many models (particularly the proprietary ones) prevents a good assessment of such models. When selecting a model, the user can benefit from comparative studies of models which were conducted earlier (8). The scope of such comparative studies was, however, limited to the models and verification data available at that time. New models and sources of verification data have appeared in the literature since then.

The documentation for many existing urban runoff models is rather poor. A surprisingly small number (~5) of models is really well documented and their documentation is continuously updated as these models are refined or modified. Even in the case of well-known models, difficulties may arise from a large number of model versions in circulation. For example, the earlier discussed SWMM model is currently available in many versions, some of which have been developed by private parties and not properly documented. Assertions have been made that some of the existing SWMM versions yield different results for identical input data.

Another problem with runoff models in public domain is the lack of model maintenance and debugging. Programming errors are typically discovered only through extensive model application under varied conditions. Unless there is a central agency looking after the correction of programming errors and distribution of these corrections, the model usefulness becomes limited and debugging efforts are needlessly duplicated.

Limitations of existing runoff models are rarely fully described and explained in the model documentation. Some models which were originally developed for rural conditions were later modified for urban conditions and vice versa. While such models have been verified in their primary use domain, their applicability to other types of watersheds is little substantiated. Two examples of such applications are urban runoff studies with the Soil Conservation Service Model and applications of urban runoff models to low-imperviousness watersheds. A further work on delineation of the applicability of individual runoff models is needed.

Finally, one should mention again the problems with model calibration and verification. When assessing runoff from newly proposed developments, a direct model calibration is not possible. The model user has to rely on transposition of calibration parameters from other similar watersheds. No systematic studies of such calibrated parameters have been conducted for developing areas. The lack of data is apparent, particularly in the case of soil infiltration rates.

6.0 KNOWLEDGE GAPS

The modelling of runoff from developing areas has evolved during the past 15 years and is still undergoing further development, particularly the modelling aspects related to water quality. A number of gaps in the knowledge of modelling of runoff from developing areas still exist and further work is required. Knowledge gaps requiring further research are discussed below.

Tools for evaluation of the effects of urbanization on the hydrological regimen are needed. Most of the models discussed earlier could be adapted for this purpose, however, at a fairly large expense. For a quick evaluation of newly proposed developments, new tools assessing the changes in the flood frequency curves areneeded. Such modelling tools should be inexpensive to use and have minimum input data requirements.

The application of the existing models requires further data on runoff from pervious urban areas. Such data could be used either for the development of new conceptual models of infiltration, or for calibration of the existing models. The application of discontinuous runoff models would further benefit from definition of initial watershed conditions. This is particularly true for simulations of runoff for design event storms which are used extensively in engineering design. Further development of simultaneous simulations of runoff conveyed by both minor and major drainage networks and the interaction between these networks are needed. Additional work on interfacing of runoff models of various hierarchies is also recommended.

Finally, flood damage curves should be developed for typical urban areas. Such information, in conjunction with the risk-based design, should be further advanced in the urban drainage design.

7.0 DOCUMENTATION

Under the Canada-Ontario Agreement, a system for distribution of runoff models and of their documentation was developed. Through this system, interested parties can obtain copies of selected computer models and the accompanying documentation. In particular, computer tapes and documentation are available for the following models:

SWMM (The latest EPA version)
CANSWMM (An earlier model version modified under the COA program)
DAM (The data analysis model described earlier and developed under COA)
STORM (A standard version)

So far, several hundred requests for this documentation have been processed. Apart from computer tapes and user's manuals, a number of pertinent research reports on model studies and applications were published under the Canada-Ontario Agreement and are available from Environment Canada.

Another source of information on urban runoff modelling is the technical bulletin CANSWMM which is jointly distributed by DOE and MOE to about 400 specialists in Canada. In co-operation with the U.S. Environmental Protection Agency, The SWMM Users Group has been organized. This group meets twice a year to discuss the latest developments in urban water resources and, in particular, the modelling of urban runoff.

Persons interested in the above listed documentation and in further information on the CANSWMM Bulletin and the SWMM Users Group should contact the author at the National Water Research Institute, P. O. Box 5050, Burlington, Ontario, L7R 4A6.

8.0 SUMMARY

Runoff from developing areas is of particular interest in connection with flood modelling because the land development dramatically alters the watershed hydrological response. During the past 15 years, a number of models for modelling of runoff from developing areas have been developed and most of the models are now fully operational. The IWD experience with these models stems mostly from the Canada-Ontario Research Program on Urban Drainage. Further development and refinement of these models is currently underway. Further improvements are needed in the areas of development of new models for assessment of the effects of urbanization on the watershed hydrological regimen, modelling of infiltration, and modelling of flood damages. Finally, a part well worth emphasizing is that models do not make decisions but, instead, provide qualitative and/or quantitative information to serve as one of inputs into the decision-making process. The purpose of modelling is insight, not numbers - "... insight into how the system behaves, to what factors it is sensitive, how man may favourably or unfavourably influence those factors, and what kind of solutions to water resource problems are likely to be technically feasible" (13).

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FLOOD FORECASTING
in the
SAINT JOHN RIVER BASIN

MATHEMATICAL MODELLING ACTIVITIES

prepared for the
INLAND WATERS DIRECTORATE
Technical Workshop
on
Mathematical Modelling Activities
related to
Flood Damage Reduction

Hull, Province of Quebec
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David A. Smith
Inland Waters Directorate
Environment Canada
Halifax, N. S.
April, 1979

ABSTRACT

The development of a flood forecasting system for the portion of the Saint John River Basin in the Province of New Brunswick is reviewed. The paper progresses, in chronological order, from the time when there was no forecasting system, through the first efforts to calibrate a mathematical model based on the SSARR program and the forecasting operations during the historic flood on record, which occurred in 1973, to the present. Emphasis is placed on the mathematical modelling activities and on recent efforts to improve the forecasting system being carried out under the Canada-New Brunswick Flood Damage Reduction Program. The major concerns of the Technical Sub-Committee for Flood Forecasting are presented and the activities presently underway, or planned, to overcome some of these difficulties are discussed.

INTRODUCTION

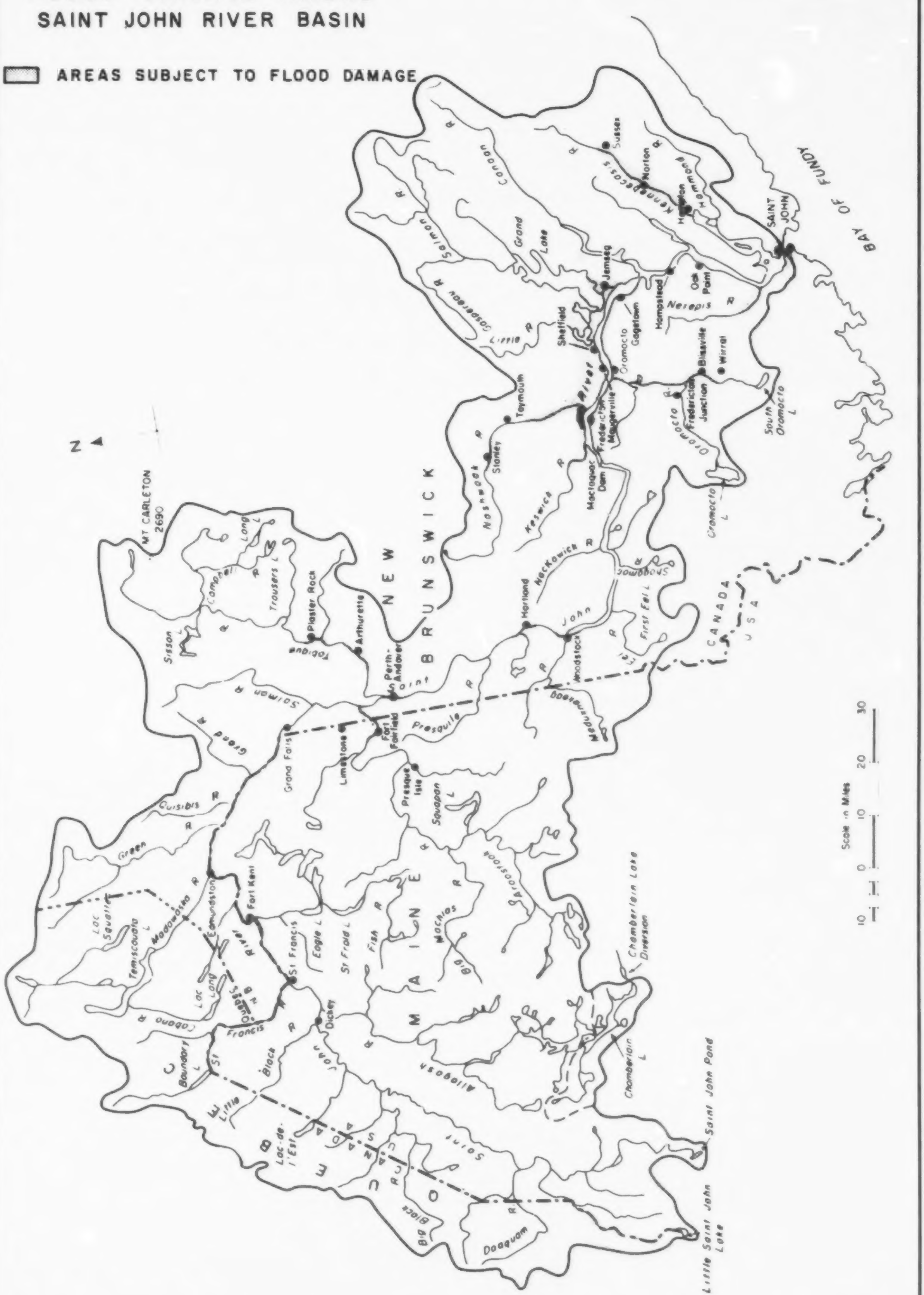
Historically, the Saint John River Basin has been subject to periodic flood damage as a result of high river flows due to rainfall and snowmelt and high stages due to ice jams. Although the monetary value of flood losses has not been large compared to that of some other parts of the country, successive floods in recent years have caused increasing damages. The three most notable examples are the floods of May 1961, February 1970 and April/May 1973 which caused damages in the order of 2.3 million dollars, 3.0 million dollars and 11 million dollars respectively.

A map of the Saint John River Basin indicating areas subject to damage is shown on Figure 1. Along the main stem of the Saint John, the principal areas of damage in Canada are the City of Fredericton and the reach between Fredericton and Jemseg where flooding of buildings and farmlands is a frequent occurrence. Overbank flooding on the Nashwaak River, the Oromocto River, and other tributaries has caused damage to bridges, highways, railways and electric power and telephone facilities.

A study[1] of the seriousness of these flood problems in the basin and the possible means of alleviating them was commissioned by the Saint John River Basin Board in 1971 as a part of their program to develop a management plan for the basin. The study was undertaken in a cooperative manner with contributions from a number of government agencies and individuals. The Atlantic Region Office of the Water

FLOOD DAMAGE AREAS

SAINT JOHN RIVER BASIN



▨ AREAS SUBJECT TO FLOOD DAMAGE
N
MT CARLETON 2690
NEW BRUNSWICK
MAINE
QUEBEC
CANADA
USA
BAY OF FUNDY
SAINT JOHN
Sussex
Norton
Hampton
Hammond R
Kennebecasis R
Canaan R
Salmon R
Grand Lake
Jemseg
Gagetown
Hampstead
Oak Point
Nerepis R
Welsford
Sheffield
Oromocto
Blissville
Fredericton Junction
Maugerville
Fredericton
Mactaquac Dam
Taymouth
Stanley
Nashwaak R
Keswick R
Little R
Gaspereau R
Oromocto L
South Oromocto L
Nackawic R
Hartland
Shogomoc R
Woodstock
First Eel L
Eel R
Meduxnekeag R
Perth-Andover
Arthurette
Plaster Rock
Tobique R
Trousers L
Campbell R
Long L
Sisson L
Salmon R
Grand R
Grand Falls
Limestone
Fort Fairfield
Presque Isle
Presquile R
Squapan L
Aroostook R
Machias R
Big Machias
Fish R
St Froid L
Eagle L
St Francis
Fort Kent
Edmundston
Madawaska R
Green R
Quisibis R
Temiscouata L
Loc Squatteck
Loc Long
Cabano R
St Francis R
Boundary L
Black R
Dickey
Allagash R
Chamberlain L
Chamberlain Lake Diversion
Little Black
Lac-de-l'Est
Saint John R
Big Black
Daaquam R
Saint John Pond
Little Saint John Lake
John R
River
Scale in Miles
10 0 10 20 30
DEPARTMENT OF THE ENVIRONMENT, AUGUST 1972

FIGURE 1

Planning and Management Branch, Inland Waters Directorate, Department of the Environment had responsibility for coordinating the study and the preparation of the study report which is entitled "The Flooding Problem in the Saint John River Basin".

One of the objectives of this study was to determine the extent to which flood damages can be reduced by a flood forecasting/warning system. If river stages can be predicted in advance and flood bulletins issued, the inhabitants of flood plains can take individual or collective action to minimize their damage. In order to determine the extent to which flood damage can be reduced by prediction and warning, data on amounts of damages were collected from government agencies, railway companies and municipalities. Since a flood forecasting system could avoid only certain types of damages, it was desirable to break down damages into avoidable losses and unavoidable losses. However, segregation of damages into these two categories was not possible. On the qualitative side, various accounts of agricultural losses involving implements, stocks and crops planted before inundation were reported as well as damages to personal moveable property. Considerable inconvenience has been suffered by those directly affected and on seven occasions loss of life has been attributed to flood waters.

During the preparation of the report a "simplified" flood forecasting method was in use by the New Brunswick Department of Fisheries and Environment for the purpose of maintaining a "watch" of the spring high flows of the Saint John River. The New Brunswick Electric Power Commission also carried out flood forecasting for the purpose of operating its hydroelectric plants on the river. Thus there was considerable interest in flood forecasts by at least two public agencies.

In the Saint John River Basin, most flood events take place in the spring when snowmelt and ice movement complicate the process of flood forecasting. Up-to-date knowledge of the accumulated snowpack, its water content, the storage capacity of the soil and the quantity, quality and state of ice on the rivers is required to develop a flood forecast sufficiently dependable to warrant interruption of normal community activity.

Existing telemark stage gauges and meteorologic networks, plus existing snow courses can, the report indicated, provide the operating data required to forecast the flow of the Saint John River. The report indicated two problems associated with operating such a forecast insofar as it is intended to reduce damages due to flooding.

The first problem was the need to improve the techniques in use at that time and to test them against some of the more severe historic floods. The need to extend the forecast to other parts of the basin was also expressed.

The second problem, and the most uncertain, was the establishment of public confidence in the forecast. If a forecast of a flood stage proved excessive, undue expense is expended in taking precautionary steps. If, on the other hand, the forecast is deficient, loss may be as heavy or heavier than if no forecasts were given. In both cases confidence is shaken.

The report concluded that the decision as to whether or not the added cost of the refinement of the forecast system was justifiable must be made subjectively because of the lack of adequate information on damages. Those associated with the study felt that such refinement was justified and that, if undertaken, it should be considered as an integral part of any flow forecasting procedure that may be developed for the multipurpose operation of the river system.

DEVELOPMENT OF THE FORECASTING SYSTEM

Towards the end of 1972, coincident with the completion of the report, renewed interest arose in the development of a system of flood forecasting using modern computerized techniques. The New Brunswick Department of Fisheries and Environment joined the New Brunswick Electric Power Commission in a search for operational forecasting programs. A computerized flood forecasting program[2] developed by the North Pacific Division of the United States Army Corps of Engineers was selected.

The forecasting program, known as the Streamflow Synthesis and Reservoir Regulation Model (SSARR) is a mathematical-hydrologic model of a river basin system which synthesizes streamflow by evaluating snowmelt and rainfall runoff. The river basin is divided into a number of sub-basins for which the basic snowmelt and rainfall runoff relationships are established. River flows are computed by routing runoff from upstream to downstream points through channel and reservoir storage. When used as a predictive tool, the model relies on meteorological forecasts of temperature and precipitation as input data. Flood forecasts can be continuously updated as recorded streamflow and precipitation data are reported. The model also has provision for handling backwater conditions such as those which exist on the Saint John River below the Mactaquac Dam but the model was not calibrated for operation under this provision right away.

The Corps of Engineers SSARR Model was obtained in early February 1973 and the task of applying this model to the Saint John River began immediately. Because of a desire to have the system operational before the 1973 spring runoff, an interagency task force was brought together from staff of the New Brunswick Electric Power Commission, the New Brunswick Department of the Environment and the Inland Waters Directorate.

The first task in model calibration consisted of making numerous computer runs for each of the 18 gauged sub-basins in the Saint John watershed above the Mactaquac Dam, using a fall rain storm event which occurred in September-October 1969. These runs provided the initial model parameters for simulation of runoff from the sub-basins.

The second task consisted of model calibration for snowmelt conditions. For this, the spring runoff of 1961 was used to obtain the parameters required for snowmelt conditions. During the watershed calibration procedure, the finest time increment utilized for inputing the data was daily mean streamflow (used to verify the accuracy of the simulation), daily precipitation and daily mean temperature determined by averaging the maximum and minimum for each day.

The third task consisted of calibration of the channel and reservoir routing component of the model. This was performed largely with the use of some time-of-travel relationships for the channel which had been derived empirically by the New Brunswick Electric Power Commission. Major reservoirs located in the headwater areas of some of the larger tributaries are drawn down well in advance of the spring runoff and are filled during the spring runoff period; therefore, calibration of these reservoirs amounted to applying essentially zero release to the tributary streams. Main stem storages at Grand Falls and Beechwood are utilized largely to create a head for run-of-river hydropower purposes and have essentially no effect on the flood hydrographs; therefore, the headponds for these two facilities were treated as part of the channel routing system. The Mactaquac Reservoir, however, while having essentially no effect on peak discharges in the Saint John River during an extreme event, such as occurred in 1973, can have a significant effect on streamflows during the rising portion of the hydrograph. This occurs when the headpond level is lowered at the dam in order to prevent upstream flooding from increased headpond slopes associated with high flow conditions. Outflows from the Mactaquac Dam were not determined by the SSARR model; rather, the inputs to the reservoir were computed and, based on operational procedures, the anticipated response was determined.

Due to the short time available, the model was calibrated using only the two historic events - one of which included snowmelt. The model was operational for flood forecasting on the Saint John River on March 26, 1973 after a two week backup period for adjustment of parameters and initial conditions.

FORECASTS DURING THE 1973 FLOOD[3]

The ability of the flood forecasting model to predict floods in advance is to a great extent dependent upon the accuracy of meteorological forecasts which are used as model input. The following is a brief description of the forecasts provided by the Atmospheric Environment Service, Environment Canada, to the Saint John River Flood Forecasting Unit during 1973.

Twice daily the Canadian Meteorological Centre (CMC) in Montreal, prepares forecasts of maximum and minimum temperatures and precipitation amounts based on 0000 GMT and 1200 GMT surface and upper air data. These forecasts are sent by teletype and facsimile network to weather offices throughout Canada. After appraisal and tailoring by local meteorologists, the information is diseminated to the public.

The CMC's temperature forecasts are based upon regression equations, applicable to individual cities. They are completely computer produced and have a valid period of either two or three days depending on the time of issue. The quantitative precipitation forecasts are also computer produced and list expected precipitation values at a number of points in a grid system encompassing most of North America. The forecast precipitation for each grid point represents the average amount expected to fall in a square with sides of 381 kilometres in length surrounding the point. The forecasting model produces large scale precipitation whenever ascending air has a dew-point depression less than a pre-determined threshold value. Smaller scale influences, including frontal precipitation, air mass showers and induced instability showers, are also incorporated into the model. The valid period for the quantitative precipitation forecasts extends 42 hours from the collection time of the data.

Special arrangements were made with the Atmospheric Environment Service to obtain subjective forecasts of temperature and precipitation beyond the period of the forecasts produced by the CMC. These forecasts were prepared by the Maritimes Weather Office meteorologists for days 3, 4 and 5. During 1973, the Fredericton Weather Office used the computer predictions of temperature and grid point precipitation to prepare forecasts for individual meteorological stations in a form suitable for input to the flood forecast model.

The accuracy of the model in forecasting streamflows is also dependent on accurate up-to-date information on actual temperatures, recorded precipitation,snow accumulation and streamflow. The network of stations used in compiling information for the model is shown on Table 1. In 1973 the network consisted of 55 temperature and precipitation stations, 22 stream gauging stations and 71 snow course stations. Data for precipitation, temperature and streamflow was compiled each morning and, during the critical period, each afternoon as well. Snow course data was received at two week intervals from cooperating agencies with periodic checks being made more frequently at some locations during the critical flood period.

The flood forecasting model was run each morning to predict streamflows for a four day period at various points along the river. As each new run was made, the previous forecast was updated to correct for changes in input forecast data and antecedent conditions in the basin.

Comparisons of recorded streamflow, precipitation and temperature with forecasts of streamflow, precipitation and temperature made one, two, three and four days in advance are shown on Figures 2, 3 and 4 for selected hydrometric stations. Examination of these figures reveals that streamflow forecasts were heavily dependent on the precipitation forecast.

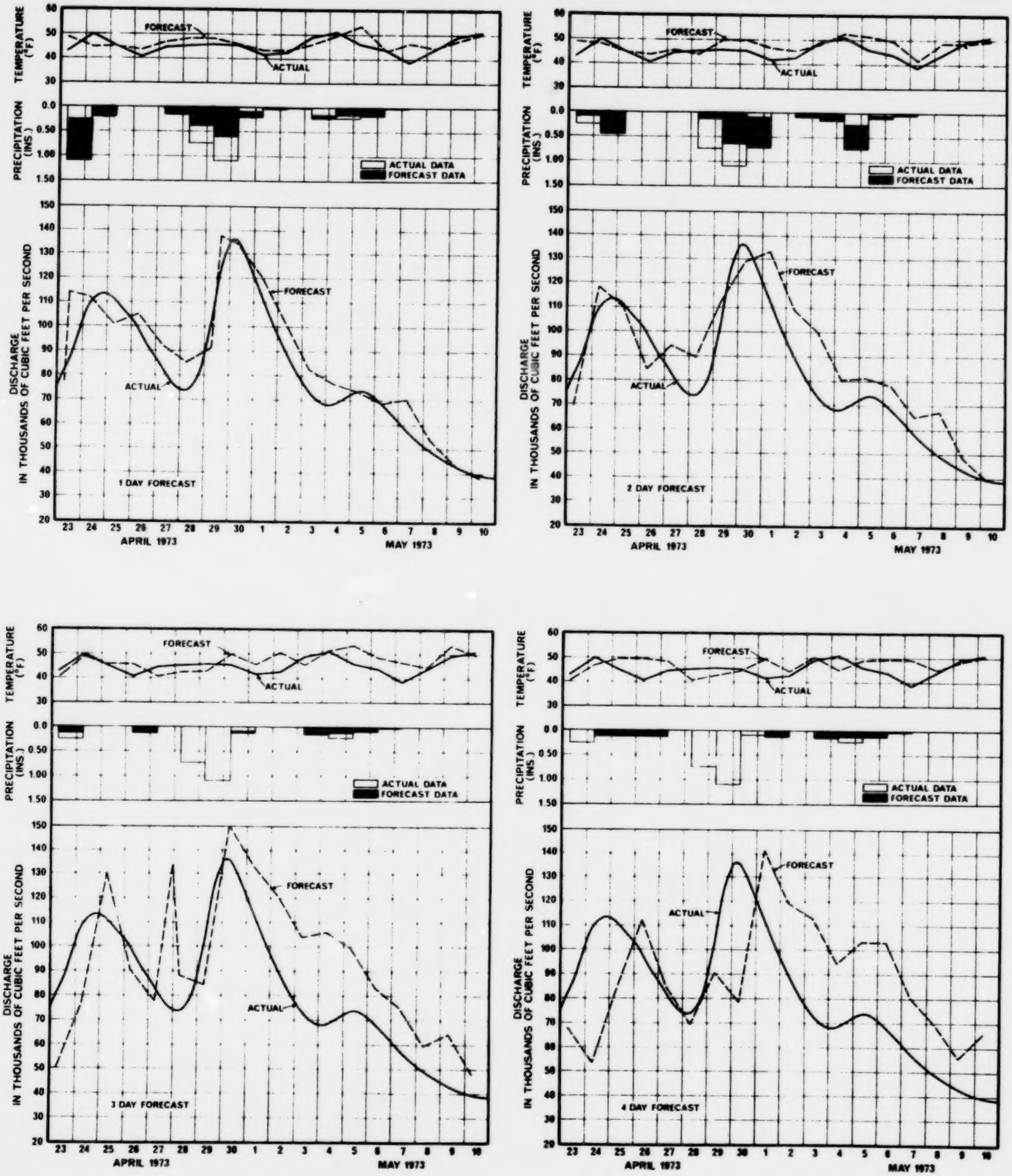
TABLE I

HYDROLOGIC DATA NETWORK USED IN FLOW FORECASTING

Type of Station	Location	Number of Stations
Temperature and Precipitation	New Brunswick	20
	Quebec	16
	Maine	19
Stream Gauging	New Brunswick	14
	Quebec	1
	Maine	7
Snow Courses	New Brunswick	41
	Quebec	7
	Maine	23

Co-operating Agencies

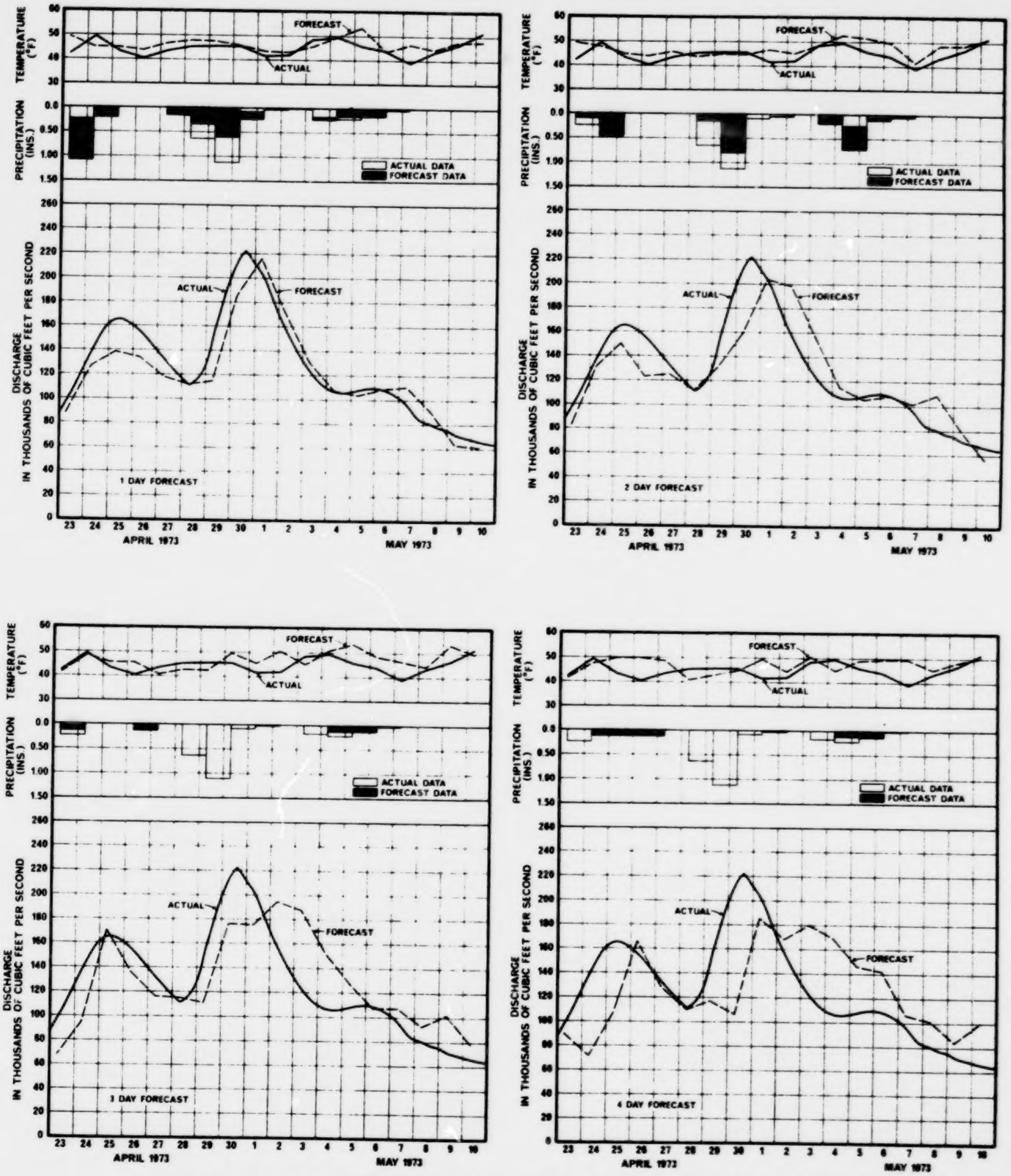
New Brunswick Electric Power Commission
New Brunswick Department of Fisheries and Environment
New Brunswick Department of Natural Resources
Quebec Natural Resources
Atmospheric Environment Service
Water Survey of Canada
Maine Public Service
Maine Parks and Recreation Commission
Maine Forest Service
National Weather Service
United States Geological Survey
International Paper Company



Comparison of Actual and Forecast Streamflow, Precipitation and Temperature - Saint John River at Fort Kent

Taken from Reference 3

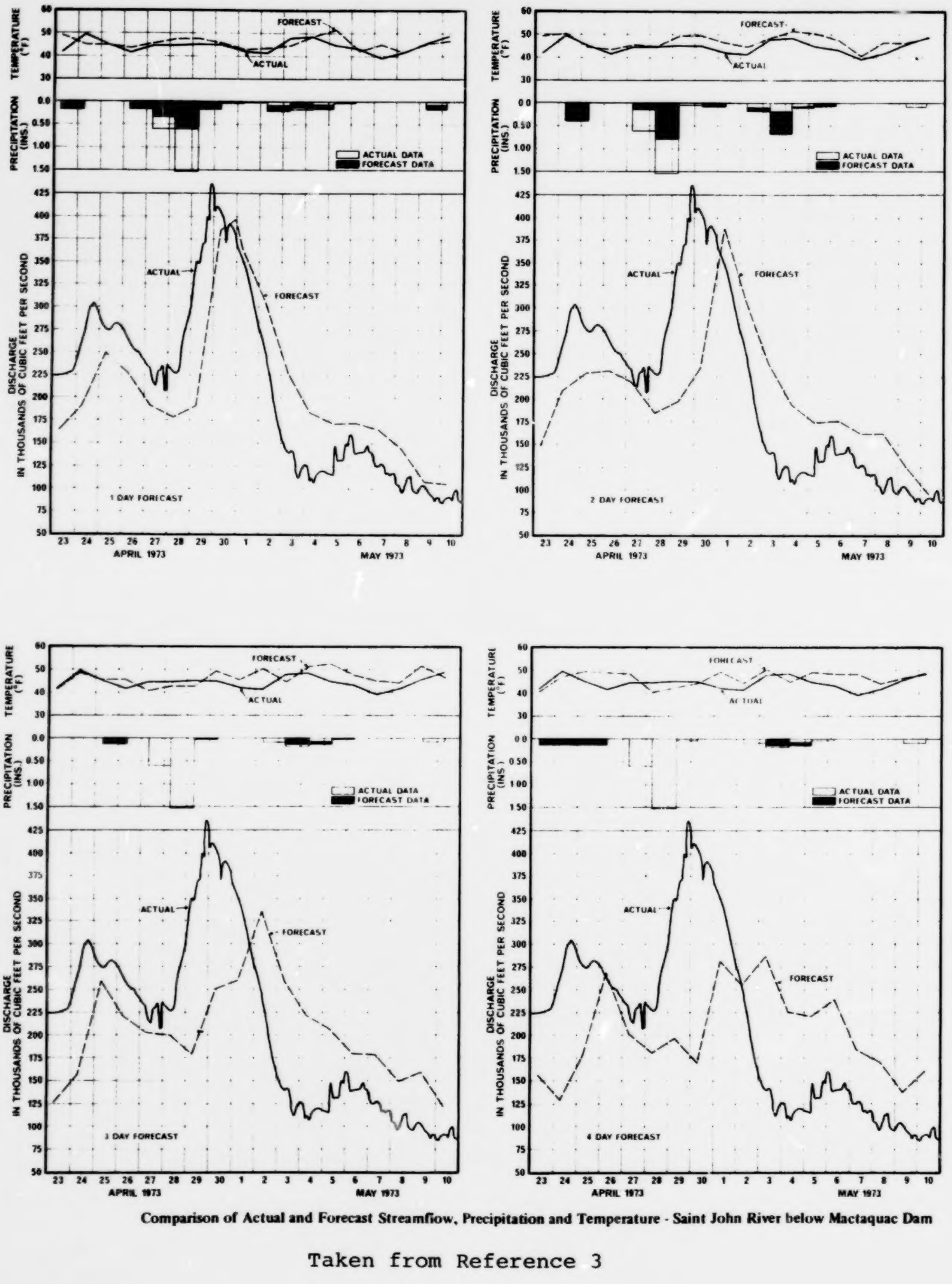
FIGURE 2



Comparison of Actual and Forecast Streamflow, Precipitation and Temperature - Saint John River at Grand Falls

Taken from Reference 3

FIGURE 3



Comparison of Actual and Forecast Streamflow, Precipitation and Temperature - Saint John River below Mactaquac Dam

Taken from Reference 3

FIGURE 4

The report "New Brunswick Flood, April-May, 1973"[3] describes in detail the record flooding which occurred in late April and early May, it's causes, progress, magnitudes, damages and the flood forecasting and emergency measures operations. The report concludes that considerable reduction in flood damage and personal hardship was achieved by the forecasting and emergency measures operations. The possibility of extensive flooding was predicted approximately three days in advance of its occurrence in the Fredericton area, while the likelihood of a major flood was predicted two days in advance.

FLOOD FORECASTING ON THE SAINT JOHN RIVER BELOW MACTAQUAC

Most of the damages caused by flooding on the Saint John River occur in the 140 km reach from just below the Mactaquac Dam to the confluence with the Bay of Fundy at the City of Saint John. Of the roughly nine million dollars damages which occurred in this area during the 1973 flood, approximately 21 per cent was to moveable items[3] even though the flood forecasting and emergency measures operations prevented a significant amount of damage. It was anticipated[3] that continued operation and improvement of the flood forecasting system and emergency measures procedures could lead to a greater reduction in the proportion of damage occurring to moveables.

During 1973 there was no adequate technique in use to forecast river stages from just below the Mactaquac Dam to the City of Saint John. The existing forecasting system provided estimates of streamflow down to the Mactaquac Reservoir but no further. Approximately 15,000 km^2 of the 56,000 km^2 total drainage area of the Saint John River lies downstream of the Mactaquac Dam. It was not sufficient to calibrate the SSARR watershed component for this 15,000 km^2 drainage area and to sequentially add the runoff from a number of sub-basins to the estimated releases from the Mactaquac Dam. A river system model was also required.

The river in this reach has a very gentle gradient. Much of the City of Fredericton, althouth it is about 120 km from the confluence, is located only seven to nine metres above mean sea level. In addition to this gentle gradient, the confluence of the Saint John River at Saint John Harbour consists of a narrow winding gorge. The combination of the construction due to this gorge and tides with a range of from seven to nine metres severely restricts the passage of river water to the Bay of Fundy during times of high discharge. Typically a large amount of the spring freshet builds up in the 670 km^2 of river channel, inlets, lakes and off-channel storage areas along this 140 km reach.

In mid-1973 the Province of New Brunswick requested the Inland Waters Directorate to extend the forecasting system to this reach. IWD agreed and decided to employ the SSARR program.[4] This decision was based on the existing use of SSARR for the upper portion of the basin and because SSARR has the ability to route river flows as a function of multi-variable relationships involving the backwater effects from tides.

The main channel, shown in Figure 5, was divided into nine sub-reaches and the off-channel storages, located north of the Trans-Canada Highway between Fredericton and Jemseg, were divided into four sub-reaches. Control stations were established at the base of the Mactaquac Dam, between the sub-reaches and at the Reversing Falls. Each of the sub-reaches were provided with a storage relationship and one or more backwater relationships.

The storage relationships were determined using four sets of aerial photograph coverages taken when the water surface was at different elevations - including the extreme levels experienced in 1973. The backwater relationships* were determined by a trial and error procedure employing the SSARR simulation program and the flow and water level data recorded in 1973. The trial and error procedure was used after initial estimates of these relationships were made using the 1973 data in conjunction with known channel geometry at the upstream and downstream ends of each reach and Manning's steady state equation.

A general criterion of no more than plus or minus 0.15 m deviation from the recorded stage hydrographs for Fredericton and Maugerville was employed in the calibration procedure.

It was not expected that the model would be able, with the emperical relationships based on a single event, to accurately simulate water levels in the lower half of the reach.

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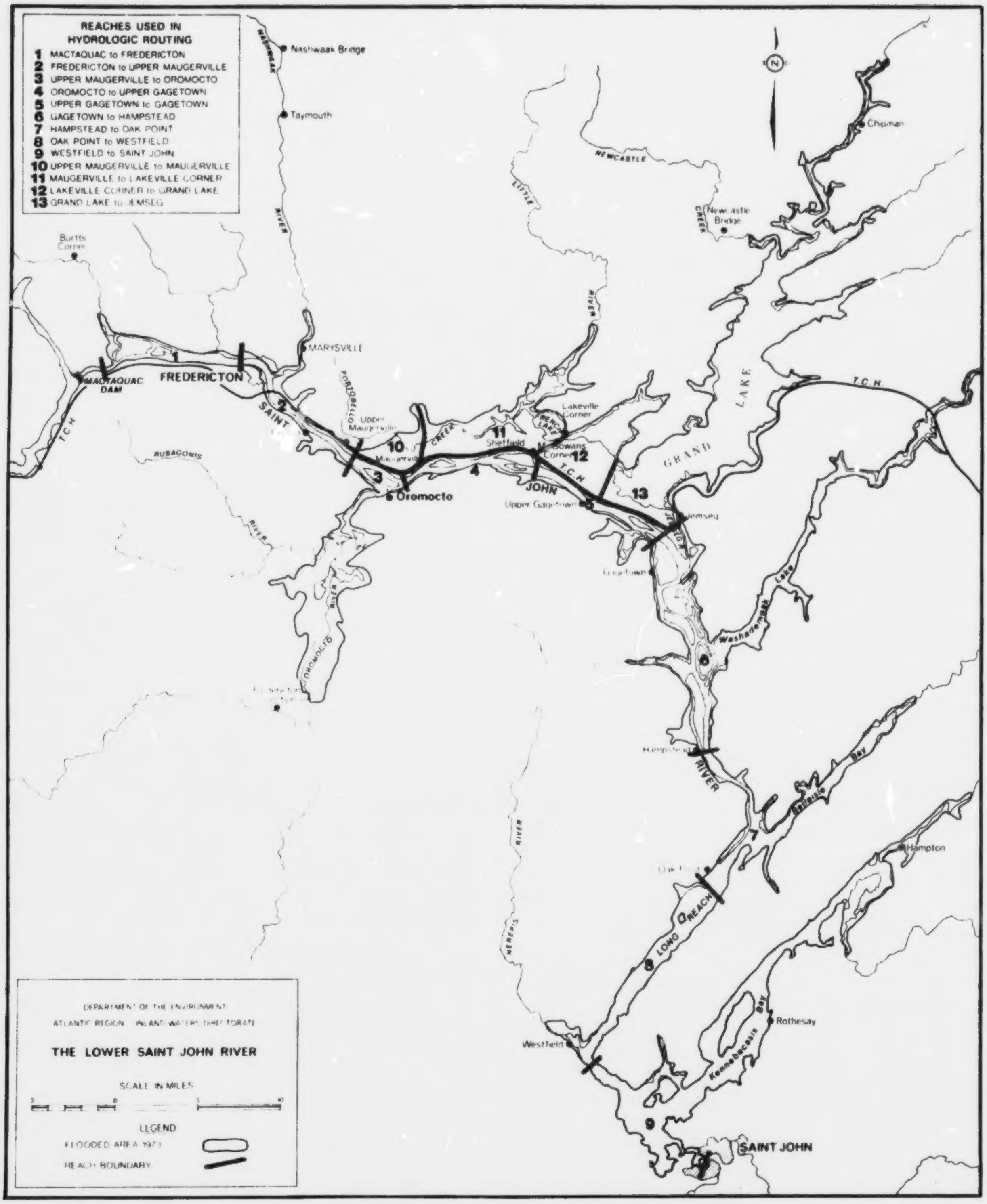
* the off-channel storages required two relationships - one to transfer flow back and forth between the main river channel and one to pass flow between themselves.


REACHES USED IN HYDROLOGIC ROUTING
1 MACTAQUAC to FREDERICTON
2 FREDERICTON to UPPER MAUGERVILLE
3 UPPER MAUGERVILLE to OROMOCTO
4 OROMOCTO to UPPER GAGETOWN
5 UPPER GAGETOWN to GAGETOWN
6 GAGETOWN to HAMPSTEAD
7 HAMPSTEAD to OAK POINT
8 OAK POINT to WESTFIELD
9 WESTFIELD to SAINT JOHN
10 UPPER MAUGERVILLE to MAUGERVILLE
11 MAUGERVILLE to LAKEVILLE CORNER
12 LAKEVILLE CORNER to GRAND LAKE
13 GRAND LAKE to JEMSEG
DEPARTMENT OF THE ENVIRONMENT
ATLANTIC REGION INLAND WATERS DIRECTORATE
THE LOWER SAINT JOHN RIVER
SCALE IN MILES
5
0
5
10
LEGEND
FLOODED AREA 1973
REACH BOUNDARY
Nashwaak Bridge
NASHWAAK
Taymouth
RIVER
Burtts Corner
MACTAQUAC DAM
FREDERICTON
MARYSVILLE
T.C.H.
SAINT
JOHN
RIVER
RUSAGONIS
RIVER
PORTOBELLO
Upper Maugerville
Maugerville
CREEK
Sheffield
Lakeville Corner
FRENCH LAKE
Bowens Corner
Oromocto
OROMOCTO
RIVER
Upper Gagetown
NEWCASTLE
LITTLE
RIVER
CREEK
Newcastle Bridge
Chipman
GRAND
LAKE
Jemseg
Gagetown
Washademoak
Lake
Hampstead
Belleisle
Bay
Oak Point
LONG
REACH
Hampton
NEREPIS
Westfield
Kennebecasis
Bay
Rothesay
SAINT JOHN
N

Taken from Reference 3

FIGURE 5

This assumption was based on the fact that the downstream control was a subjectively estimated relationship to simulate the discharge to Saint John Harbour as a function of the water levels just above the gorge and the elevation of the water in Saint John Harbour. No field measurements of flow through the Falls were available to verify the relationship.

Verification of the model was based on a simulation of the response of the lower reach of the Saint John River to the 1972 spring freshet and on its performance in subsequent forecasting operations.

In addition to the channel routing component, the response of the watersheds of the Saint John River below Mactaquac had to be simulated. Unfortunately there were only four streamflow gauges in operation recording the runoff from just 22 percent of the drainage area below Mactaquac. The New Brunswick Electric Power Commission undertook the task of calibrating these four sub-basins using the same procedures and events employed for the sub-basins above Mactaquac. In addition the data for 1973 was used.

Runoff from the remaining 78 percent of the drainage area below Mactaquac was estimated by dividing the area into a number of sub-basins and pro-rating the flows for the gauged sub-basins on the basis of the ratios of the respective drainage areas. This is the same procedure which was employed for the forecast model upstream of Mactaquac.

OTHER IMPROVEMENTS TO THE FORECASTING MODEL

As each spring freshet passed, the calibrations of the 22 gauged sub-basins were checked and adjusted based on the data collected that spring. In 1975, a team from IWD, the New Brunswick Electric Power Commission and the New Brunswick Department of the Environment re-calibrated six of the sub-basins based on the data for a total of ten spring events.

Improvements were also made in the information system supplying the input data required to operate the forecasting system. From 1975 on, the National Weather Service of the United States supplied the forecast centre with estimates of the snow covered area for each of the 22 calibrated watersheds. These estimates were obtained using a geostationary weather satellite. The New Brunswick Department of the Environment developed a procedure to massage the data on water equivalents from the snow course network into iso-lines. A number of telemark units were installed at strategic streamflow gauges to facilitate the daily initialization of the forecast model. Several satellite platforms and DARDC* units were installed by the United States Army Corps of Engineers, Environment New Brunswick and IWD.

The Water Survey of Canada installed several temporary stage gauges in the off-channel storage area in the lower reach of the Saint John River. These gauges provided useful information of the exchange of water between the main channel and the four large off-channel storages.

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* DARDC - Device for automatic remote data collection on command via telephone coupler or radio link. Developed by the U. S. National Weather Service.

The New Brunswick Department of the Environment and the University of New Brunswick[5] joined forces and constructed a physical model of the gorge which forms the outlet of the Saint John River. This model provided a relationship to determine discharge as a function of the tidal elevation in Saint John Harbour and the water surface elevation just above the gorge.

Flood forecasting operations continued to be provided as they had since 1973 from the offices of the New Brunswick Electric Power Commission on an ad hoc basis with staff from the Power Commission, the New Brunswick Department of the Environment, IWD and the Atmospheric Environment Service (when a severe meteorological condition warranted). After each spring freshet period, forecasting operations were terminated and the staff returned to their routine functions in their respective agencies.

Development and operation of the flood forecasting system continued in this uncoordinated manner until 1976. On March 31, 1976, the Governments of Canada and New Brunswick entered into a General Agreement Respecting Flood Damage Reduction. The General Agreement provided for several sub-agreements to contribute funds, management and technical advice to provide flood risk maps for flood prone areas, to study areas with special flooding problems and to establish and operate a flood forecasting centre and develop a flood forecasting system. In anticipation of an early signing of a flood forecasting agreement for the Saint John River Basin, an ad hoc technical committee was established to coordinate the ongoing activities. The "Agreement Respecting Flood Forecasting for the Saint John River Basin" was not signed, however, until August 31, 1977.

ACTIVITIES UNDER THE FLOOD DAMAGE REDUCTION PROGRAM

The Agreement Respecting Flood Forecasting[6] has two distinct phases. The first phase, scheduled for completion on March 31, 1980, includes the establishment and staffing of a flood forecasting centre, the provision of interim forecasts using existing procedures and techniques, the development and calibration of a flood forecasting system and the documentation of formal flood forecasting procedures. The second phase, scheduled for completion at the termination of the Agreement on March 31, 1982, consists of providing forecasts with the newly developed system and the undertaking of any further development work required. The total amount of funds authorized by the Agreement and to be shared equally between Canada and New Brunswick is $600,000.

When the Agreement was signed, a work plan was prepared[7] by the Technical Sub-Committee on Flood Forecasting and approved by the Steering Committee. A Director* was appointed for the Flood Forecasting Centre and a flood forecasting technician was hired. The New Brunswick Department of the Environment assigned a hydrologic engineer and a meteorological technican to work at the Centre. The New Brunswick Electric Power Commission and the Inland Waters Directorate agreed to provide technical assistance as required during forecast periods.

A search was commenced for suitable accommodations; however, it was not until May, 1978, that the Centre was established in the newly re-located offices of the New Brunswick

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* the Program Manager for the Canada-New Brunswick Flood Damage Reduction Program, Mr. J. G. Lockhart was appointed Director of the Flood Forecasting Centre on a part-time basis.

Department of the Environment in Fredericton. Forecasting operations during the spring of 1978 were once again provided from the offices of the New Brunswick Electric Power Commission.

The Agreement Respecting Flood Forecasting provided the required vehicle to coordinate the activities directed at improving the forecasting system. Interest has been expressed in having the capability of the forecasting system extended to provide forecasts of floods at practically every flood-prone area indicated in Figure 1. It was thought that the system could be improved by recalibrating either the existing SSARR-based model or a model based on the National Weather Services' River Forecasting System (NWSRFS). Improvements were also thought possible in the data management procedures and in the data collection network.

The Technical Sub-Committee for Flood Forecasting has a number of concerns about the present system. These include:

(1) The inability of the present SSARR-based watershed model to accumulate snow and to accurately simulate watershed response to a rain-on-snow event, a common occurrence during the spring runoff period on the Saint John River. These is also interest in improving the snowmelt relationships perhaps using data from small research watersheds to improve on the base temperature of 0°C. At present if the forecast maximum and minimum temperatures are say +5°C and -5°C, a mean temperature of 0°C is computed and no melt is simulated.

(2) The way in which runoff from unguated areas is determined in the calibration and forecasting modes is not

adequate. While the existing technique of prorating the runoff on the basis of the ratio of drainage areas with a "hydrologically representative" gauged watershed is fairly simple, nearby gauged areas are often not representative of the characteristics of the unguaged drainage areas and the technique ignores the meteorological inputs which can be uniquely assigned to the unguaged areas.

(3) The use of daily mean streamflow data, daily precipitation and daily mean temperature data to calibrate the watershed component of the model.

(4) The lack of a simple and inexpensive technique to provide accurate estimates of the water equivalent of the snowpack in each of the sub-basins. Present research into techniques to measure the water equivalent of snow remotely using gamma radiation may prove successful and applicable.

(5) The lack of a technique to indicate the variation in soil moisture over the basin. This applies to situations when the Flood Forecast Centre is required to initialize the model prior to the commencement of an anticipated heavy rainfall event during the late spring, summer or fall seasons.

(6) The use of daily mean streamflow data to "calibrate" the channel routing component of the model. For the various reaches into which the Saint John River was divided for routing purposes, the time-of-travel under high runoff conditions is only a matter of hours.

(7) A significant amount of flood damage occurs as

the result of ice jamming on the Saint John River as well as its many tributaries. The present forecasting system has no means of foreseeing such occurrences.

(8) A large amount of error can be attributed to the meteorological forecasts of precipitation. Improvements in meteorological forecasts for days one, two and three would be reflected directly in the flood forecasting accuracy.

(9) The stage forecasts issued for the Saint John River from Jemseg downstream to Saint John by the existing SSARR-based model are not sufficiently accurate. The present model requires a large amount of effort to calibrate the emperically based backwater relationships and also requires a reasonably accurate discharge relationship at the downstream control.

(10) A dynamic hydraulic model of the Mactaquac reservoir is required in order to accurately forecast the response of the headpond to predicted inflows and specified operating procedures and to accurately determine releases to the lower reach of the Saint John River.

(11) The flood forecast system should, when economically feasible, be extended to a number of flood prone areas on the main channel and on the major tributaries.

(12) The meteorological network available for calibration and updating of the forecast model is not adequate. There are a number of locations in the basin, generally remote, where climatological stations would be beneficial.

(13) The streamflow gauge network in the basin

downstream of the Mactaquac Dam is inadequate. Barely 22 percent of the drainage area is metered.

(14) Coordination with appropriate agencies in the United States is necessary to solve difficulties with the snow survey network. In the past few years the number of active snow courses in the United States' portion of the basin, roughly 35 percent of the total drainage area, has been decreasing steadily. There is also considerable (up to one week) delay in receiving the data from these agencies in addition to difficulties encountered in scheduling all snow course surveys for roughly the same day.

(15) Backup computer facilities and transmission equipment is required in the event of a computer system breakdown or power outages.

With these concerns in mind and with a centre established and staffed, the Technical Sub-Committee has begun the task of developing an improved flood forecasting capability for the Canadian portion of the Saint John River Basin.

Economic and engineering investigations are underway to determine the feasibility of extending forecasts to other flood prone areas. As a result of these studies, forecasts are being provided for a number of additional locations during 1979 and a field program has been established for the collection of data required to extend forecasts to more locations during 1980.

The Technical Sub-Committee has established desired flood forecast accuracies[9] for flow of ±10 percent and for stage of ±.15m for days 1 and 2. The desired accuracy for the prediction of time of peak flow and peak flood stage has been set at ±6 hours. For all flood prone locations, a two-day warning is considered the minimum lead time required for significant response and the accuracy criterion for stage is to take precedence over the accuracy criterion for flow.

An appraisal[10] of the accuracy of the day 1 forecasts has been completed. This appraisal indicates that forecasts of flow are improving with time; however, they are not yet meeting the desired accuracy limit of ±10 percent. The day 1 forecasts of stage in the Saint John River from Fredericton down to Saint John are generally within the desired accuracy of ±0.15m. Analyses have not been undertaken for day 2 as there is sufficient indication, at the day 1 level, of a need for improvement to the system.

A comparison of the NWSRFS model, which is now being implemented in various River Forecast Centres throughout the United States, and the SSARR model was undertaken by IWD[8] Atlantic in order to determine which model should be applied to the Saint John River Basin. The NWSRFS has a number of interesting features; however, since it was determined to be not fully transposable, the Technical Committee decided to continue using the SSARR model. Progress in the development of the NWSRFS system will be monitored, however, and the Flood Forecasting Centre will encourage the NWS to utilize the system on the United States' portion of the Saint John River in order to compare performance.

The major concern that the Technical Sub-Committee has with the SSARR model is its inability to accumulate snow and its inability to simulate the rain-on-snow event. This portion of NWSRFS is being added to SSARR by the Corps of Engineers and the new version should be available in the near future. The Technical Sub-Committee has made arrangements to obtain the new version when it is fully tested and documented.

The Sub-basin boundaries have been redefined to incorporate several additional gauged areas, to provide a more hydrologically-based division of the ungauged portion of the basin and to provide discharge summing points at a greater number of locations on the main stem.

Plans have been made to calibrate the gauged sub-basins using three-hour streamflow data, hourly precipitation values and daily mean temperature data in conjunction with the eight, three-hour melt distribution values permitted per day. All sub-basins will be recalibrated using three or four non-snowmelt events and six or seven snowmelt events.

A decision has not yet been made on the way in which the runoff from ungauged basins is determined. In the upper portion of the basin most of the drainage area falls within calibrated watersheds; however, in the lower portion of the basin barely 22 percent of the drainage area is within calibrated sub-basins. Several techniques are possible, including a pseudo-calibration procedure in which watershed parameters are assigned based on the parameters for hydrologically representative, calibrated sub-basins.

The river routing component for the main channel will be calibrated using hourly streamflow data for about five to ten events.

The DWOPER one-dimensional, dynamic routing model developed by Dr. D. Fread of the United States National Weather Service has been successfully applied, under the Flood Risk Mapping Agreement by MacLaren Atlantic Ltd., to the task of producing the 1:20 and 1:100 year recurrence interval flood profiles for the Saint John River from the Mactaquac Dam to Lower Jemseg. During the course of this study, a number of flow measurements were made by the Inland Waters Directorate, under a variety of tide conditions in Saint John Harbour, to determine a discharge relationship for the Reversing Falls, the downstream control for the model. In addition to the improved rating curve produced for the Reversing Falls, this model made greater use of existing and newly surveyed cross-sectional data for the 140 km lower reach of the Saint John River. This calibrated model has been obtained by the Flood Forecasting Centre with the intention of employing it to provide improved stage forecasts for the Saint John River from the Mactaquac Dam to Saint John.

In order to facilitate the exchange of information necessary for the provision of adequate forecasts for the Saint John River Basin, a letter of intent was prepared by the Technical Sub-Committee at the request of the Steering Committee for the Canada-New Brunswick Flood Damage Reduction Agreement. This letter has been signed by five Canadian agencies including the Quebec Department of Natural Resources, the New Brunswick Department of the Environment, the New Brunswick Electric Power Commission, the Atmospheric Environment Service and the Inland Waters Directorate. The letter was forwarded recently to three United States Agencies including the United States Geological Survey, the National Oceanic and Atmospheric Administration and the United States Army Corps of Engineers. It is hoped that regular meetings of these agencies can be convened. Possible benefits to flood forecasting efforts for the Canadian portion of the Saint John River Basin include the stabilization and enhancement of the snow course network and coordination of the surveys; the calibration and operation of the NWSRFS for the Saint John River Basin down to Fort Kent, Maine, and improvements to the network of satellite data collection platforms in the upper portion of the basin.

The preceding discussion indicates that the Technical Sub-Committee is responding to a number of the concerns expressed on pages 23 to 26. Progress to alleviate concerns with respect to the provision of spatial estimates of soil moisture and snowpack water equivalent will depend on the success of current research programs. Improvements in ice jam and meteorological forecasting techniques should be of significant benefit to flood forecasting.

It is anticipated that the recalibration of the watersheds and channel routing components of the model will be complete by March 1980 in time for the spring runoff season.

The flood forecasting system for the Saint John River Basin will probably never be completely developed; however, as there will no doubt always be something about the system which can be improved.

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DATA NETWORK DESIGN FOR WATERSHED FORECASTING

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Flood Damage Reduction
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INTRODUCTION

Data network or observation network design is a large scale long-term optimization project. The objective is to obtain or define streamflow characteristics anywhere within the network to a certain level of accuracy. The optimum solution to the objective function is restricted by physical, social and economic constraints. The best possible data or a specified accuracy are required for a specified or minimum cost and effort.

Hydrologic data networks depend upon the collection purpose. Data networks are classified and subdivided in specific and numerous ways. One subdivision has individual stations labelled as primary, secondary or special purpose. Another classification is in accordance to the level of interference of man on the watershed regime. Stations may be defined as being on natural, transitional or modified regime streams. Each subgroup within the overall network has its own optimum techniques of planning and data estimating. Data network density is directly related also to the data use and a level of estimation accuracy. Hydrologic data required for flood forecasting varies from data required for hydraulic design or annual basin yield, just to name a few.

The network purpose to be addressed here is specific to supporting an operational flood forecasting procedure.

Inland Waters Directorate, Atlantic, Experience and Performance

A Flood Damage Reduction Agreement was signed in March 1976 with the Province of New Brunswick. A subagreement was established to support the development of a Flood Forecast Centre for the Canadian portion of the Saint John River. Within the five year agreement, three years are for the development phase, while the last two years are for the forecast operation.

I.W.D. has been involved with flood forecasting on the Saint John River for many years. A forecasting model began to be applied in early 1973. The Steering and Technical Committees within the FDR program have made various decisions since the general program began in 1976. Specifically, it has been decided to apply the Streamflow Synthesis and Reservoir Regulation (SSARR) Model to forecast high and flood flows on the Saint John River to a point at which flows enter the Mactaquac Hydroelectric Dam headpond. The SSARR Model is also applied in the lower basin, Macaquac Dam to the mouth at the Bay of Fundy, to generate inflows that are subsequently routed by the Dynamic Wave Operational Model (DWOPER).

In 1972, the World Meteorological Organization Commission on Hydrology initiated the beginning of pilot studies in international basins. From a number of possible basins within Region IV, Canada and the United States selected in 1974 the Saint John River Basin for a World Weather Watch pilot study. I.W.D., Atlantic, was involved in various Task Force reports specifically related to the hydrologic data network.

A new involvement in the FDR program is now developing with the signing of an agreement with the Province of Nova Scotia.

The Data Network
Saint John River

Hydrological data collection began in the Saint John River Basin about 1918. Historically, there have been sixty-seven(67) hydrological stations in the Canadian basin area and eleven(11) stations in the United States portion. There have been seventeen(17) stations along the main stem of the Saint John River. Currently, there are thirteen(13) main stream gauging stations.

The location of the hydrological stations is illustrated on the WMO WWW project map entitled "Gauging Stations". At the end of the 1975 calendar year, there were forty-six(46) hydrological stations in New Brunswick, four(4) stations in Quebec and eleven(11) stations in Maine.

The SSARR Model calibration in 1973 used thirty-four(34) subbasins upstream of the Mactaquac Dam. The availability of meteorological and hydrological data for calibration established this number of subbasins. The combined upper and lower basin models now operationally use forty-three(43) hydrological stations.

The WMO WWW Project also considered networks established to gather other types of parameters. Maps illustrating the climatological, snow survey, water quality and groundwater networks have been produced. The cooperative nature of these networks is shown in Figure I of this paper.

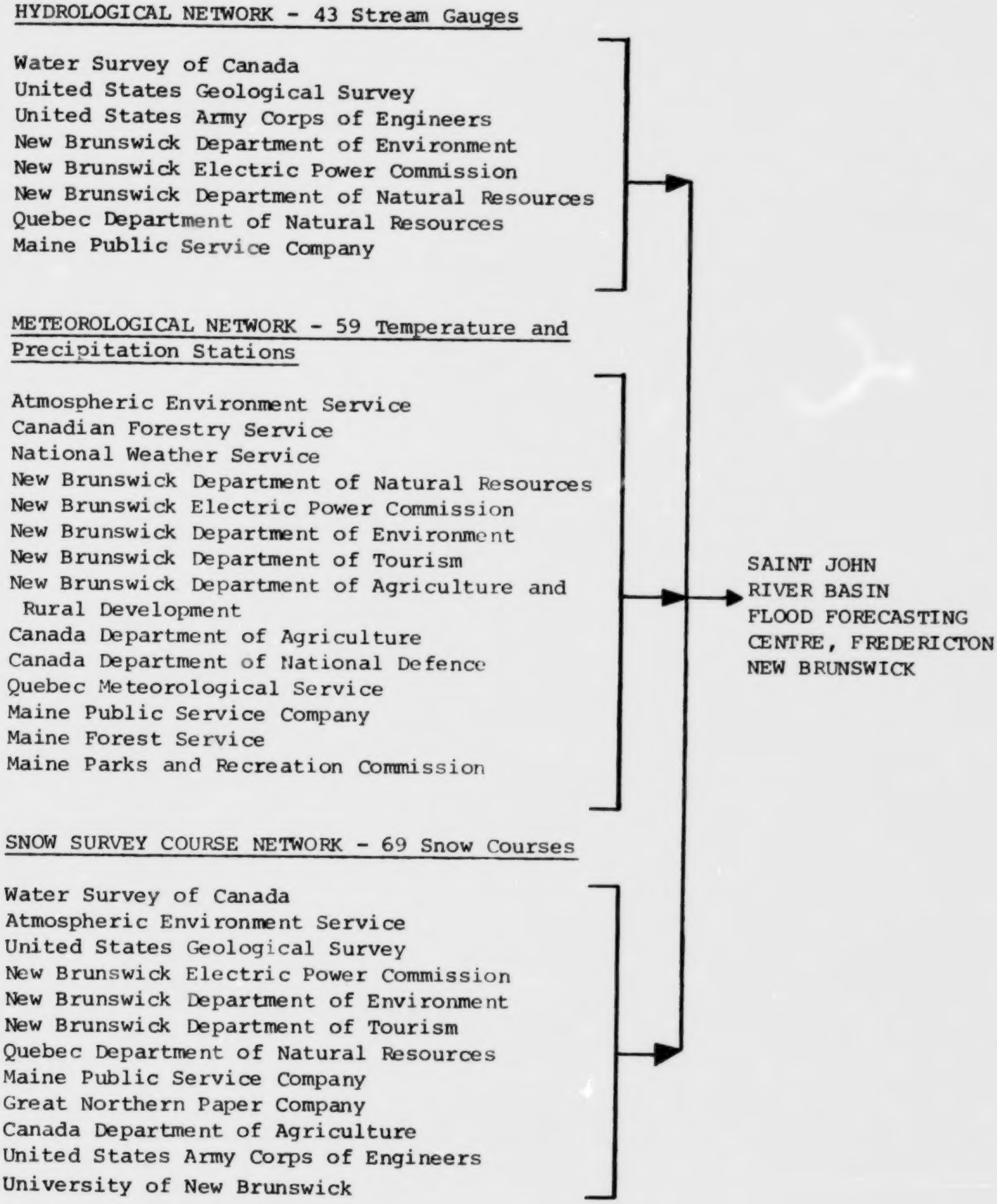
Network Requirements

The type of model applied to promote flood damage reduction will dictate, to a large degree, the data network required to support the program.

In most FDR applications the data network will already exist at varying levels. The special needs of the FDR program have to be clearly identified so that the value of the existing network may be assessed. The decision was made to continue using the SSARR Model and apply the DWOPER Model where the unsteady, non-uniform flow conditions prevail in the lower basin.

Judgement of a physically-based mathematical hydrologic simulation model's performance may be quantified by determining how well the model can be calibrated to characterize the hydrologic responses of individual watersheds, channel reaches, lakes and reservoirs and coordinate these responses to simulate the reaction of the system as a whole.

An appraisal was requested to evaluate the adequacy of the existing stream gauging network as it provides input to the SSARR computer program to forecast spring freshet flows and flood flows during other periods throughout the year.



HYDROLOGICAL NETWORK - 43 Stream Gauges

Water Survey of Canada
United States Geological Survey
United States Army Corps of Engineers
New Brunswick Department of Environment
New Brunswick Electric Power Commission
New Brunswick Department of Natural Resources
Quebec Department of Natural Resources
Maine Public Service Company

METEOROLOGICAL NETWORK - 59 Temperature and Precipitation Stations

Atmospheric Environment Service
Canadian Forestry Service
National Weather Service
New Brunswick Department of Natural Resources
New Brunswick Electric Power Commission
New Brunswick Department of Environment
New Brunswick Department of Tourism
New Brunswick Department of Agriculture and Rural Development
Canada Department of Agriculture
Canada Department of National Defence
Quebec Meteorological Service
Maine Public Service Company
Maine Forest Service
Maine Parks and Recreation Commission

SNOW SURVEY COURSE NETWORK - 69 Snow Courses

Water Survey of Canada
Atmospheric Environment Service
United States Geological Survey
New Brunswick Electric Power Commission
New Brunswick Department of Environment
New Brunswick Department of Tourism
Quebec Department of Natural Resources
Maine Public Service Company
Great Northern Paper Company
Canada Department of Agriculture
United States Army Corps of Engineers
University of New Brunswick

SAINT JOHN
RIVER BASIN
FLOOD FORECASTING
CENTRE, FREDERICTON
NEW BRUNSWICK

Figure I
Agencies Supporting The Observing System
(1977)

The network constraint to characterize high or flood flows is deliberately emphasized. A basin response to high flows is much different from that of low flows. High or flood flows tend to mask or diffuse many non-homogeneous characteristics of watersheds. It follows from this that the monitoring network for large flows is much different from that required to monitor mean and low flows.

A practical approach to this problem was taken by asking the following question.

What is the role of a stream gauge in support of the SSARR model application?

There would appear to be three roles played by a stream gauge.

These roles are:

1. *The calibration role, where the stream gauge supplies observed discharge and/or water level data against which the results of the computed outputs are judged.*
2. *The start-up role, (initial conditions) where the stream gauge supplies observed data to permit the model to be adjusted to a best representation of the current hydrologic processes.*
3. *The monitoring role, (verification) where the stream gauge supplies observed data against which the forecast results may be judged once the model is operating.*

An examination of these roles indicates that one, the first role, has the lead when developing a forecasting procedure. Streamflow data to provide initial conditions and verification of forecasted flows are subsequent needs after a calibration has been completed and forecasting is operational. Enough gauges should be available for fulfill the calibration role to the extent that a sufficiently accurate calibration can be obtained. A sufficiently accurate calibration is relative to the needs of the forecast user, the data base and other factors.

The errors of past flow and level forecasts on the Saint John River have been well quantified (Pol, 1977) and the required levels of accuracy at various forecast locations along the basin have also been quantified as a task report for the WMO WWW Project (Hansen, 1977). These two documents have provided the Program Manager and Technical Committee of the Canada-New Brunswick Flood Forecasting Agreement with information to answer a third question. Does the current calibration provide a 'sufficiently accurate' model? There exist several sources of error which will effect a forecast.

The SSARR Model, based on calibrated parameters rather than statistical correlations, uses numerous tributary runoffs to generate forecast point discharges. The computed outflow is dependent upon the calibration of a number of partial drainage basins. This applies both to the calibration mode and the operational mode.

In the parameteric type SSARR Model, the number of subbasins employed for calibration certainly affects the output accuracy. Recalibration using various subbasins would indicate an optimum number, all other parameters held constant. This is a type of sensitivity analysis. The problem at hand in terms of the data network is to determine the number and location of operational reporting stations. There is a point where an increase in the number of stations reporting to the Forecast Centre provides no significant improvement in forecast accuracy.

With the specific network requirements in mind,the use of a stream gauge for calibration purposes leads to the following line of thought:

Streamflow is a residual of precipitation; however, variance of streamflow over months and years is much greater than the precipitation, which is its source. Precipitation, the dominant cause of the time variation of streamflow has regional homogeneity; thus, streams in a certain area usually exhibit similar patterns of flow. This means that the pattern of one stream may be judged from other streams and therefore not all streams require long-term records for the model calibration and operation. At the same time, different catchments respond differently to the same precipitation. This variation, known as spatial, is due to physiographic, land use and cover properties. This variance is, nevertheless, relative to climatic conditions and under flood flow conditions the climatic factors override all other features.

This line of argument points towards the identification of regions having precipitation homogeneity. During large streamflow events, a hydrologic station in an area of similar snowpack melt and rainfall should adequately reflect all flow conditions within the homogeneous area. For flood forecasting purposes, the assessment of hydrologic data network be based firstly upon precipitation homogeneity while for low flow, the hydrologic network assessment be based on physiographic homogeneity.

Station density versus accuracy has been used to assess various networks. However, this analysis indicates that the error of estimate based on the number of stations may be misleading. Different configurations of the same number of stations may result in different standard errors. Spatial distribution is as important as density. One station in a representative area tells more than many stations in a homogeneous area.

Main stem stations must be carefully considered. Stations too close to each other are redundant. If the difference in runoff between two stations on the same river is not greater than the limit in the error of measurement, then the additional station is not justified. Main stem gauging sites below the confluence of gauged tributaries are not justified. On the other hand, tributary discharge is not well estimated by subtracting two main stem stations if it is suspected that the error may represent a large portion of the tributary inflow. Each tributary flow monitoring station to satisfy the optimum need must provide independent information. This is essential to the SSARR Model. Watershed simulations need calibration verification and operational check points. Main stem stations, although not providing independent data, are required for the verification and checking of routing parameters.

The SSARR Model Application requires at least a gauge on larger tributaries and that these gauges be located as close as possible to the main stem confluence. The variations within watersheds are integrated during high flows, thus, only tributary inflow to the main stem is important. If however, there is a location within a large tributary for which a forecast is required, then secondary stream gauging stations may be necessary.

The data network in the Saint John River Basin has improved to support the FDR program. Two new stream gauging stations have been implemented. A station on the Nerepis River monitoring an area of 292.7 km^2 (113 mi^2) now provides natural regime runoff data for the extreme southwest basin just above the Saint John River at Saint John New Brunswick. This station was established within the Canada New Brunswick Co-operative Agreement for water quantity monitoring. No real-time data is now available from this station. The newest hydrological station is on the Grand River on the east side of the Saint John River between Grand Falls and Green River. The natural regime data from this 175.3 km^2 (67.7 mi^2) basin area is to be available on a real-time basis at the Flood Forecast Centre. The New Brunswick Electric Power Commission identified this station as an important contribution to flood wave magnitude and timing.

Concurrently, consideration has been given to real-time VS non-real-time needs and to manual VS computer reduction and storage of data. The data network must respond when required as the FDR program moves into the operational phase. The practical considerations such as access and cost of real-time response must also be kept in mind as a data network is designed for watershed forecasting. It is realistic to consider a data network as a dynamic system. Periodic reviews are necessary because of changing user needs and technology advances. The degree to which a data network is optimized varies as the constraints change with time.

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OTTAWA RIVER

COMPARISON OF THE HYDRO QUÉBEC AND SSARR MODELS

by

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to be held May 23 - 25, 1979 in Hull, Quebec

OTTAWA RIVER - COMPARISON OF SNOWMELT ROUTINES IN QUEBEC HYDRO AND SSARR WATERSHED MODELS

Flood forecasting in the Ottawa River Basin has become important in recent years because of the flooding which occurred on the Gatineau River in 1974 and the yearly concern in Montreal that high flows from the Ottawa and St. Lawrence Rivers could cause property damage. Because of this, Water Planning & Management Branch calibrated the SSARR hydrometric simulation model and in the years 1977-78, real time forecasts of watershed and streamflow discharges were made, with generally acceptable results. In 1978, improvements to the output format and the preparation of a special watershed map made it much easier for the users to understand and interpret the data. Manual adjustment of the melt rate in the watershed calibration also greatly improved overall performance of the model.

The principle users of the forecasts are the Quebec and Ontario Hydro companies and the Ministries of Natural Resources. The federal Department of Public Works is also involved, since they control two of the major reservoirs in the system. The flood control interests of the Provincial and Federal governments oppose the power development objectives of the Hydro companies. As a result, an Ottawa River Planning Committee was formed to represent all groups and to deal with flood forecasting and other problems.

Quebec Hydro, as part of its overall responsibility for generating power within the province, has developed a hydrometric model of its own. This model is very similar to SSARR. In 1978, the two models were run simultaneously and a comparison was made based on the results of five watersheds. The choice of these watersheds tended to favor the Quebec Hydro model. In the comparison, three Quebec Hydro watersheds were judged by the Committee to have performed significantly better and based on this, the Quebec Hydro model was chosen for the 1979 forecast run. The fact that some manual adjustment of the SSARR model is needed to optimize its performance was also a consideration in this decision. As a result, the Water Planning & Management Branch watershed forecasting activities on the Ottawa River were abandoned. This decision also meant that considerable effort was needed to expand the Quebec Hydro model to include all of the Quebec and Ontario watersheds and to adapt a stream routing routine.

The SSARR and Quebec Hydro Models

The differences between the two models is small. The Quebec Hydro model has two or three refinements in the snow melt routine and a somewhat different method of routing the surface, subsurface and base flow to produce a final watershed discharge.

SSARR has two snow melt options, the generalized equation, which requires more data than is normally available on a real time basis, and the temperature index equation. The latter option was used. The equation is:

$$M = (T_A - T_b)\ R\ \left[\frac{PH}{24}\right]$$

where:

T_A = Period temperature at the median elevation of the melting snowpack. The average temperature is used in the Ottawa River model but T_A could also be the period minimum or maximum.

T_b = Base temperature - the melting temperature which is established during calibration

R = Melt rate - this coefficient was used to adjust the model to optimize the 1978 runoff

PH = Period Length in hours - (12 hrs in the WPM model)

The corresponding equation in the Quebec Hydro model is:

$$F = FAC\ (T_{max} - T_B)\ SA$$

where:

FAC = Melt factor (see below)

T_{max} = Maximum daily temperature

T_B = Base temperature at which melt occurs

SA = the proportion of the basin surface covered with snow

In order to take into account the variation of albedo with snow, the melt factor increases with the number of consecutive days without snow:

$$FAC = FKM\ (1.15 - 0.4e^{-0.38T})$$

where:

FKM = melt rate - as in the SSARR model

T = number of consecutive days without snow

This factor varies between 0.88 for one day to 1.15 after 12 days.

The snow area factor:

$$SA = GG - GG \cdot FT/SNETM$$

where:

FT = Total melted snow (FT = sum of the melt from each previous period)

$SNETM$ = Total accumulated snow on the watershed

GG = calibration factor (1.4 to 1.6)

The SSARR handles the snow area with two options. One is very similar to the Quebec Hydro approach and the other, called the snow band option, divides the basin into elevation dependent bands and melts the snow in each at a rate dependent on the daily temperature adjusted for band elevation. The snowband option is used in the Water Planning & Management Branch SSARR model. In this option, the total snow area changes step by step as the snow in each band is melted.

A further feature of the Quebec Hydro model is that a fixed percentage of the melt does not enter the runoff equations, and if freezing occurs, some of this melt finds its way back into the snow pack accumulation.

The two models handle the routing of the melt through the three types of runoff to the final watershed discharge somewhat differently. In the SSARR model, a table of SMI vs Percentage of Runoff, generated as part of the calibration, determines the amount of melt actually made available for runoff. The SMI is adjusted each period based on a monthly evapotranspiration index (ETI), the amount of precipitation plus melt, and a factor which reduces ETI on rainy days. The water available for runoff is distributed to surface, subsurface and base flow using tables, again generated as part of the calibration. Each type of flow is then routed through a series of artificial lakes using the equation:

$$T_S = \frac{KTS}{Q^n}$$

where:

T_S = time of storage

KTS = calibration coefficient

Q = discharge

n = calibration coefficient between -1 and +1

Without going into detail, the Quebec Hydro model uses SMI in much the same way to determine the amount of melt available for runoff, and then employs a method based on unit hydrographs to route the surface, subsurface and base flows to a final summed watershed discharge.

Model Calibration

An important feature of the Quebec Hydro model is its semi-automatic calibration procedure. This routine allows the watershed to be quickly calibrated over one or more years of data. The coefficients to be calibrated must have initial values chosen between established limits and adjustments must be made to insure that a global optimization is achieved.

The SSARR model, on the other hand, is calibrated manually. Usually an autumn storm is used to calibrate the routing and then a spring runoff to determine the remaining watershed coefficients.

A significant difference in the approach taken by Quebec Hydro in the calibration of the model is in the choice of watersheds. In the SSARR model, 20 of the 26 calibrated watersheds are calibrated to a gauge on what is essentially a non-regulated stream. The Quebec Hydro model, on the other hand, organizes their watershed so that the calibration is done against the calculated natural inflow into a reservoir. From the results given below, it is seen that Quebec Hydro has been able to achieve good results doing this where, in fact, five of the seven SSARR watersheds which involve reservoirs, performed poorly.

The SSARR model, when it was originally calibrated, was intended primarily for determining potential flooding situations in Montreal. As a result, the calibration emphasized an accurate final discharge rather than individual watershed accuracy. Some of the watershed errors tended to cancel out. As soon as the model was put into use, the performance of the individual watersheds became important and for this reason, the need for some recalibration of the SSARR model is indicated by the final results.

The Quebec Hydro model in 1978 was calibrated for only 10 watersheds, all in the province of Quebec.

Comparison of Model Results

In 1978 the Quebec Hydro and Water Planning & Management Branch SSARR models were run simultaneously, and a comparison of results for five watersheds was made to determine which model should be used for the 1979 spring flood. To make this comparison, the results of each forecast day over the entire forecast period were expressed in terms of a performance coefficient. This coefficient was first used during the calibration of the SSARR model because of the difficulty in subjectively judging the model's performance between calibration runs and between watersheds. The C_F coefficient is the weighted average of three other coefficients. These are: the Nash coefficient, which measures the general quality of the run with equal weight, extended to high and low discharges; the Pointe coefficient, which weighs the performance in terms of increasing discharge, and finally a Volume coefficient which measures the quality of the water balance. Values of C_F between 0 and 0.5 mean an excellent run, while values greater than 1.25 are considered not acceptable.

The watersheds chosen for the comparison were Dozois, Kipawa, Baskatong, Paugan and Carillon. The choice of these tended to favor the Quebec Hydro model because in the case of Baskatong and Paugan, only part of the watershed had been calibrated. Kipawa is one of the 30 non-calibrated watersheds within the SSARR model which have to be related to a calibrated watershed in terms of area. The other 21 SSARR watersheds, some of which performed well, and the remaining five from Quebec Hydro were not included in the comparison

A problem in comparing the two models is that the weather forecast used by each organization were not the same. The Quebec Hydro forecast originated with AES but were massaged by a meteorologist, who in fact lead the forecast team and developed the Quebec Hydro model. A second problem which was mentioned earlier, is that the SSARR model was calibrated for overall rather than individual watershed performance.

Table 1 gives the results of the comparison. Each day the daily forecasts were grouped together and the C_F coefficients calculated. The results show close similarity at Carillon and Dozois, although the first five days at Dozois favor the Quebec Hydro model. The Quebec Hydro watershed at Kipawa is more superior, but as mentioned, this is an uncalibrated watershed in the SSARR model. If the Quebec Hydro Kipawa results are compared to Dumoine, it is seen that this watershed is better over the first five days. Similarly for Baskatong, the Quebec Hydro watershed is better, even when compared to Ceizur, the calibrated SSARR watershed used to calculate Baskatong. Paugan again favors the Quebec Hydro Paugan watershed but Maniwaki, a calibrated SSARR watershed within the Paugan area is better.

Table 2 gives all of the SSARR plus the five Quebec Hydro watersheds. Coded values have been used instead of actual C_F coefficients and the watersheds have been ranked in approximate decreasing performance. Only seven SSARR watersheds are seen to be completely acceptable, and taking individual forecast days, 43% have C_F values greater than 1.25 and are therefore unacceptable.

The last five watersheds in Table 2 are associated with reservoirs, and these are all seen to have unacceptable performance. This contrasts sharply with the Quebec Hydro watersheds which are all calibrated in terms of calculated natural inflow into reservoirs.

General Conclusions

The five Quebec Hydro watersheds presented for comparison certainly performed well and fell in the top third of the ranked SSARR watersheds. How Quebec Hydro's remaining five performed is not known.

From the results, it is clear that calibration improvements are required to at least half the SSARR watersheds, especially those involving reservoirs. It is equally clear, however, that the SSARR model is capable of producing good forecasts.

An improved or massaged weather forecast would also help the SSARR performance because the Quebec Hydro forecasts remained fairly uniform over the 10-day forecast period, while the performance of the SSARR watersheds generally decrease.

TABLE 1

Total Performance Coefficients C_F for Hydro Quebec, SSARR Comparison

Forecast Day

		1	2	3	4	5	6	7	8	9	10
Dozois	Q.H. Model	.62	.68	.68	.69	.74	.81	.84	.90	.94	.94
	SSARR	.85	.88	.85	.80	.78	.80	.82	.88	.95	1.03
Kipawa	Q.H. Model	.54	.57	.64	.67	.68	.66	.62	.66	.74	.80
	SSARR	.73	.77	.82	.98	1.14	1.32	1.55	1.79	1.98	2.08
Baskatong	Q.H. Model	.36	.50	.60	.64	.64	.67	.76	.85	.91	.90
	SSARR	.61	.76	.77	1.33	1.79	1.95	2.20	2.22	2.01	1.84
Paugan	Q.H. Model	.51	.52	.56	.61	.66	.71	.79	.85	.87	.88
	SSARR	1.27	1.28	1.23	1.21	1.25	1.35	1.44	1.44	1.39	1.29
Carillon	Q.H. Model	.45	.50	.55	.60	.63	.64	.63	.66	.70	.72
	SSARR	.45	.51	.57	.65	.71	.75	.73	.69	.68	.71
Ceizur	SSARR	.48	.67	.83	1.08	1.28	1.50	1.64	1.64	1.48	1.42
Maniwaki	SSARR	.35	.40	.44	.48	.59	.66	.75	.81	.87	.87
Dumoine	SSARR	.31	.44	.49	.56	.69	.85	.98	1.13	1.27	1.45

TABLE 2

C_F Coefficients Ranked in Approximate Decreasing Watershed Performance

	Forecast Day									
	1	2	3	4	5	6	7	8	9	10
Petite Nation	1	1	1	1	1	2	2	2	2	3
Maniwaki	1	1	1	1	2	2	2	2	2	2
Rouge	1	1	2	2	2	2	2	6	3	3
Coulonge	1	1	2	2	2	3	3	3	3	3
Q.H. Baskatong	1	2	2	2	2	2	2	2	2	2
Q.H. Carillon	1	2	2	2	2	2	2	2	2	2
Noire	1	2	2	2	2	3	3	3	2	2
Q.H. Kipawa	2	2	2	2	2	2	2	2	2	2
Q.H. Paugan	2	2	2	2	2	2	2	2	2	2
Q.H. Dozois	2	2	2	2	2	2	2	2	2	2
* Dozois	2	2	2	2	2	2	2	2	2	3
Petawawa	2	2	2	2	2	2	2	3	3	3
Dumoine	1	1	1	2	2	2	2	3	4	4
York	1	1	2	2	2	3	3	4	4	4
Chelsea	1	2	2	2	2	3	3	4	4	5
* Bark Lake	2	2	2	2	2	3	4	5	6	6
Mont Laurier	2	2	3	3	3	3	4	4	4	4
Maganasipi	1	2	2	2	3	4	4	4	4	4
Mattawa	1	2	2	3	3	4	4	4	4	4
Blanche	2	2	2	3	3	4	4	5	5	5
Ceizur	1	2	2	3	4	4	5	5	4	4
Mississippi	1	2	2	3	5	6	6	6	6	6
South Nation	2	2	3	4	5	5	5	4	5	5
Kinojevis	2	2	4	4	4	3	3	3	3	3
Bonnechere	2	4	6	6	6	6	6	6	6	6
* Cabonga	5	4	4	4	4	4	4	4	4	5
* Mistinikon	5	5	6	6	6	1	6	6	6	5
* Kiamika	6	6	6	5	5	5	6	6	6	6
* Lady Evelyn	6	6	6	6	6	6	6	6	6	6
* Mitchinamecus	6	6	6	6	6	6	6	6	6	6

Range of C_F Values

1 = .0 to ≤ .5
2 = .5 to ≤ 1.0
3 = 1.0 to ≤ 1.25
4 = 1.25 to ≤ 1.5
5 = 1.5 to ≤ 2.0
6 = > 2.0

Note: value ≥ 4 represents unacceptable performance

* - indicates a SSARR watershed calibrated against calculated reservoir inflows

SHORT-TERM AND LONG-TERM HYDROMETEOROLOGICAL EPOCHS*

By Walter Nemanishen, P. Eng.

ABSTRACT

This paper, prepared to achieve some of the objectives of the Inland Waters Directorate Workshop, is not intended to provide a comprehensive review of hydrometeorological epochs**. It briefly describes how and why Water Survey of Canada (Calgary Office) became involved in this problem and the benefits which have consequently accrued to ongoing drought-flood studies. Scientific evidence is becoming stronger to indicate that extra-terrestrial factors, such as solar activity and lunar cycles, influence atmospheric and oceanic circulation which then combine to produce prolonged droughts or floods in specific locations of the Northern Hemisphere. The studies of several prominent world scientists are highlighted to show how their findings can be used to improve IWD studies of Western Canadian flood problems. A case history is presented to illustrate the difficulty of assigning correct return periods for rare flood events and the valuable contribution that runoff modelling can make in these cases.

I. INTRODUCTION

The extreme variation in annual runoff of Prairie streams makes efficient water management and flood control difficult. Compounding the runoff variation problem is the tendency for a series of either dry or wet years to occur sequentially as demonstrated by South Saskatchewan River and Qu'Appelle River annual runoff plotted in Figure 1. A cursory inspection of these graphs immediately suggests that there are cycles in the annual runoff but their periodicity is uncertain.

It was never the original intention of Water Survey of Canada (Calgary Office) to investigate hydrometeorological epochs. The first step towards the study of drought-flood epochs was unknowingly made in 1972 when the ILB method of water supply forecasting was discovered during the Prairie Provinces Water Board streamflow forecasting study. A brief review of this forecasting method is provided in this paper and the subsequent investigations undertaken to provide a scientific rationale for the ILB method's ability to provide a quantitative index of short-term (i.e. up to six months) hydrometeorological conditions within the South Saskatchewan River Basin. The ILB forecasting studies were later amalgamated with ongoing WSC drought-flood studies to isolate some of the more important linkages or teleconnections among oceanic, atmospheric, and extra-terrestrial factors responsible for short-term and long-term hydrometeorological epochs in the Plains region east of the Canadian Rockies.

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* Paper presented at the Technical Workshop: Modelling Activities Related to Flood Damage Reduction (Inland Waters Directorate, Environment Canada): May 23-25, 1979.

** The term epoch is used rather than cycle as it more accurately describes the persistence of certain runoff sequences observed on Prairie streams.

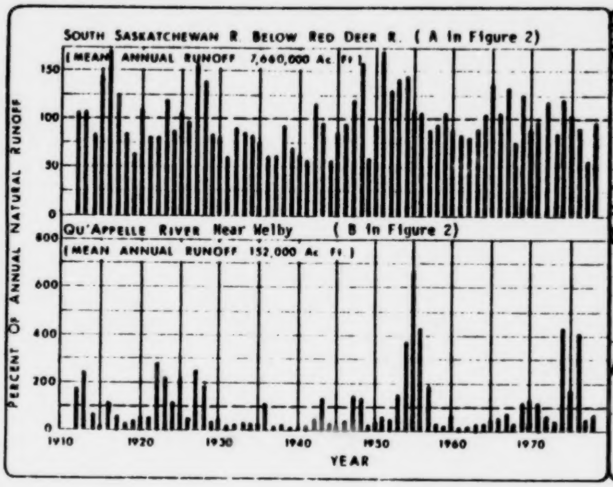

Figure 1: Variation in annual runoff of two prairie streams.

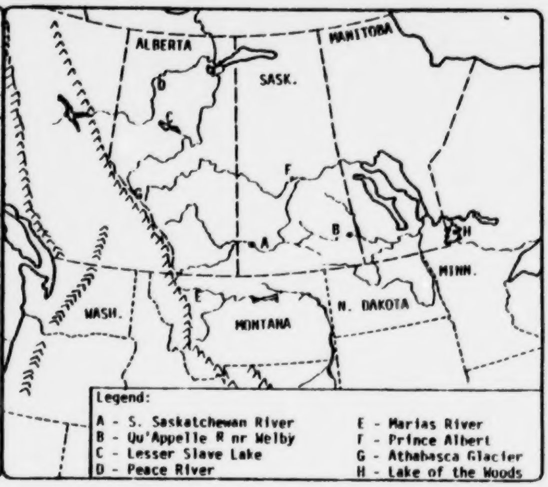

Figure 2: Location map of places mentioned in this paper.

Subsequent to the signing of the streamflow Apportionment Agreement in 1969, the three Prairie Provinces and the Federal Government reconstituted the Prairie Provinces Water Board (PPWB). This agency then proceeded to fund the five-year PPWB streamflow forecasting study conducted by Water Survey of Canada, Calgary Office, and Atmospheric Environment Services. During the course of this study, an accurate, short range water supply forecasting procedure was discovered. It was designated the Intergrating Loss Basin (ILB) method and is described fully in a report by Water Survey of Canada to the PPWB. A brief description of this method is provided next to explain why this discovery, when combined with some ongoing flood studies, lead to limited investigation of short-term and long-term hydrometeorological epochs.

II. INTEGRATING LOSS BASIN (ILB) FORECASTS

Prior to PPWB study, Rodgers (1964) had developed a series of spring and summer water supply forecasting equations for the South Saskatchewan River. These equations used winter precipitation, summer precipitation and winter baseflow as the independent variables in the forecasting equations. These equations had correlation coefficients and average error of about 0.90 and 13% respectively between the recorded and forecasted runoff volumes.

During the PPWB study, Rodgers' procedures were extended to include other hydrometeorological variables. Significant correlations were revealed between runoff and winter temperatures, snowpack evaporation-sublimation, autumn,and spring evaporation rates. Incorporation of these variables improved the accuracy of the forecasting equation but individual annual errors could still vary about ±25%.

Through an unusual series of circumstances, it was discovered that there was a remarkable fit (See Figure 3) between the runoff forecast residuals (i.e. discrepancy between the observed and predicted runoff) and the early snowmelt runoff of small plains streams, denoted Intergrating Loss Basin (ILB) streams. As the ILB runoff data is usually available by April 1st annually, the values were introduced as the fourth independent variable in Rodgers' equations. Table 1 shows the errors and pertinent statistical parameters between recorded and forecasted runoff for forecasts made using only the data available up to the date of computation. Average values were used for the independent variables entering the equation subsequent to the computation date. The remarkable feature of the ILB method was that the April 1st forecast could predict the next seven months of runoff with an average error of less than ten percent compared to 18% for the equation without the ILB term.

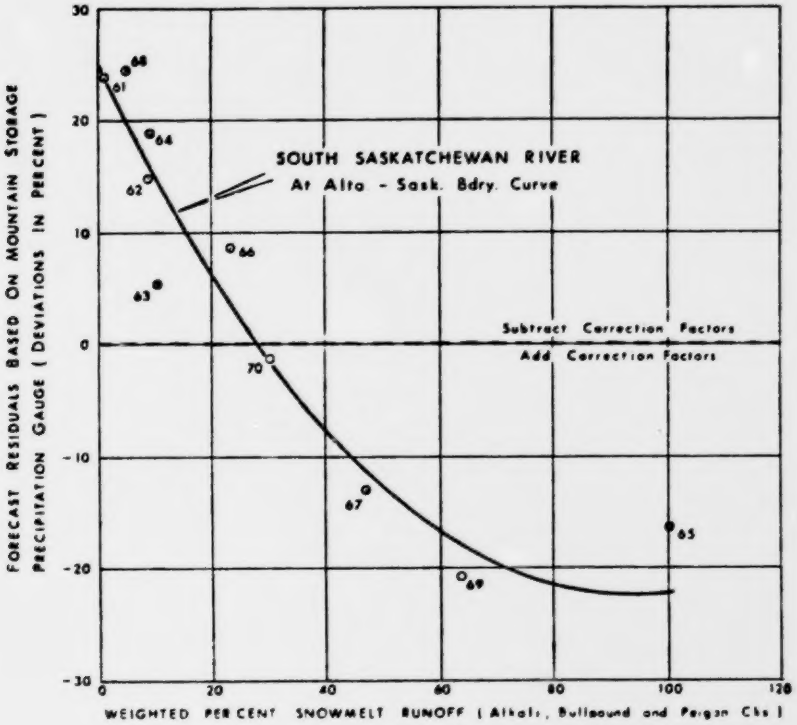

Figure 3:
Graph of Forecast Residuals versus snowmelt runoff of small prairie streams.

Table 1: Comparison of Water Supply Forecasts* Updated Monthly South Saskatchewan River below Red Deer River

X_1 Values	Forecast Date	Without Integrating Loss Basins						With Integrating Loss Basins (X_4)					
		R	S Kilo Ac.-Ft.	S/M	Average % Dev.	Worst Forecast		R	S Kilo Ac.-Ft.	S/M	Average % Dev.	Worst Forecast	
						%	Year					%	Year
Mountain Storage Gauges	March 1	.547	1,798	.242	17.74	28.98	1962	.469	1,897	.255	18.98	29.48	1962
	April 1	.605	1,710	.230	17.42	32.31	1962	.967	545	.073	6.00	11.81	1962
	May 1	.599	1,721	.232	16.89	-25.01	1965	.981	421	.057	4.53	7.06	1961
	June 1	.546	1,800	.242	17.70	26.72	1961	.977	457	.062	4.97	- 8.67	1963
	July 1	.735	1,457	.196	14.43	-21.40	1965	.992	269	.036	2.96	- 5.11	1968
	Aug. 1	.716	1,501	.202	15.08	21.51	1961	.992	263	.035	3.04	5.45	1961
	Sept. 1	.695	1,544	.208	15.05	-20.38	1969	.994	226	.030	2.67	4.28	1961
	Oct. 1	.680	1,437	.193	15.23	-23.56	1969	.995	213	.029	2.55	6.21	1961

x_1=a index of winter precipatation

The WSC report to the PPWB recommended that the ILB forecasting method be adopted even though no detailed scientific explanation could be provided for its unusual predictive capability. It was also recommended that additional studies be conducted on the ILB method to eventually link the ILB parameter to April to October runoff.

By 1973, it was apparent that the ILB method was providing a quantitative index of a short-term climatic epoch. For example, warmer than average winter months tended towards low snow packs, frequent chinooks and dry-early spring: typical winters have been 1940-41, 1960-61, 1972-73 and 1976-77. Conversely, a cool rainy autumn could be followed by cold, heavy snowfall winters and late, wet springs. The available evidence suggested that the hydrometeorological factors contributing to a short-term epoch (i.e., 6 to 12 months persistence) tended to organize by early autumn and continue until the following summer.

The WSC recommendation that runoff forecasts use the ILB method was adopted by the PPWB. The responsibility of calculating South Saskatchewan River forecasts was delegated to WSC Calgary Office. With this assignment came not only the challenge and responsibility to provide all user agencies with accurate seven month forecasts, but the opportunity to continue studies on hydrometeorologic epochs.

III. KNOWLEDGE ACQUIRED FROM FLOOD STUDIES

Interspersed with streamflow forecasting and other hydrological activities, WSC Studies and Hydrology Section has been researching and writing flood reports.

In the Lesser Slave Lake area north-central Alberta, severe flooding occurred in 1971. Before a flood report could be written, floods occurred again in 1973, 1974, 1975, 1976, and 1977. In preparation for writing one flood report to describe this epoch of floods, the historic record lake levels were reduced to present Geodetic Survey of Canada datum. As large gaps existed in the records, the U.S. Army Corps of Engineers' SSARR program was used to compute approximately 180 months of missing lake levels in the record from 1915 to 1977. In addition, this computer program was used to calculate the net daily inflows to the lake for the entire period of record. These inflows were required for flood frequency analyses and separating snowmelt from rainfall runoff.

When the complete 1914 to 1977 sequence of Lesser Slave Lake levels was plotted (Nemanishen and Cheng, 1978) an inverse relationship was noted between north-central Alberta-Saskatchewan runoff and U.S.A. plains' droughts. This feature was described in a paper to the third International Hydrology Symposium at Colorado State University (Nemanishen, 1977). In this paper, it was suggested that a terrestrial factor was the most dominant linkage between solar related phenomema and hydrometeorological epochs east of the Rocky Mountains. Furthermore, it was theorized that this linkage, 1) operated mainly in a random mode, 2) could be biased for prolonged periods and, 3) could respond to extra-terrestrial perturbations if these exceeded threshold values and coincided with favorable switching times.

Using the above theory as an *a priori* framework, the scientific papers accumulated during the PPWB study were reappraised to determine where WSC's meagre knowledge could productively interface with that published by experts in this field. A brief review of the research most relevant to IWD flood responsibilities is provided next.

IV. POSSIBLE CAUSES OF LONG-TERM HYDROMETEOROLOGICAL EPOCHS

Solar and Lunar Connections

Although there are numerous published papers on correlations between solar phenomena and hydrometeorological observations, the works of Smirnov (1969 and 1974), Brier (1968), and Brier and Bradley (1964) are briefly highlighted here as their findings are significant to interpreting some of the results presented later in the case study.

Smirnov's (1969) initial studies were confined mainly to the Volga and Dnieper River which discharge into the Caspian and Black Seas, respectively. His studies were influenced by A.A. Girs who had identified a series of epochs in the atmospheric circulation effecting precipitation in the two river basins. Using Girs' data, Smirnov found that the Volga River discharge was decisively linked to specific atmospheric circulation epochs:

> "...........*the average flow of the Volga at Volgograd during the epoch of westerly circulation W (1900-1928) it was 8400 m 3/sec, during the easterly epoch E (1929-1939) it was 6800 m 3/sec, during the northerly epoch N (1940-1948) it was 8000 m 3/sec, and during the E + N epoch (1949-1960) 7700 m 3/sec. It must be noted also that all the years with extreme flow fluctuations in the Volga are confined to periods of anomalous development of processes of the W and N, or E types. Maximum flow was observed during zonal circulation W and minimum flow during meridional circulation E, when a high-pressure ridge persisted over the entire eastern part of the European USSR and West Siberia.*"

Smirnov analyzed sunspots and long-period tide producing forces in studying runoff variations (see Figure 4) and found useful relationships.

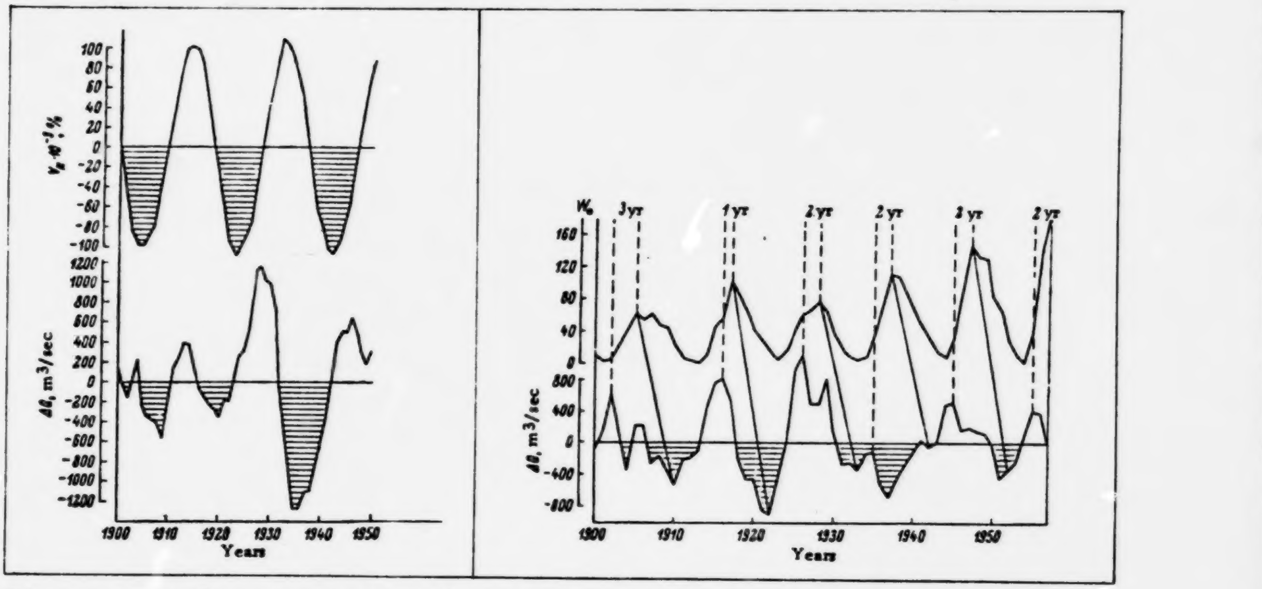

Figure 4: Left: Comparison of variations of the Long-period part of the tide producing force at latitude $\varphi = 50°$ with flow anomalies of the Volga at Volgograd.
Right: Comparison of mean annual relative sunspot numbers with the flow anomalies of the Volga at Volgograd (reproduced from Smirnov, 1969).

Of the four epochs listed by Smirnov, the one of most interest to our drought-flood studies is the 1929-1939 one. During this particular decade, the Volga River basin experienced precipitation deficits similar to the Plains region of North America, as evidenced from runoff data for the South Saskatchewan and Qu'Appelle Rivers (Figure 1). A significant factor of this decade is that there was an excess of precipitation north of latitude 54° in Canada and possibly in Europe also. An explanation for this anomaly is to be found in Brier's work.

According to folklore, it's possible to predict summer rains or droughts by observing the crescent of the new moon during winter months. Depending on tilt of the crescent, it will "retain" or "spill" water. These positions foretell of impending droughts or floods. Referring to such folklore, Dr. Glenn W. Brier and Donald A. Bradley, experts on lunar synodical periods and precipitation, observe (1964):

> ".......*There are isolated instances of provincial folklore which are sometimes provocatively accurate*........"

Brier's and Bradley's study (1964) is very comprehensive but only one of his examples is reproduced here as Figure 5 to illustrate the relationship between precipitation and lunar synodical period. It is apparently based on 16,057 record dates of maximum 24 hour precipitation at 1544 U.S. weather stations between 1900 and 1949. Of this relationship, they state:

> "*Harmonic analysis of the "national curve" shows a dominant wave having a frequency of two* cycles per synodical month, with maximum enhancement of precipitation at synodic decimals of 0.11 and 0.61.*"

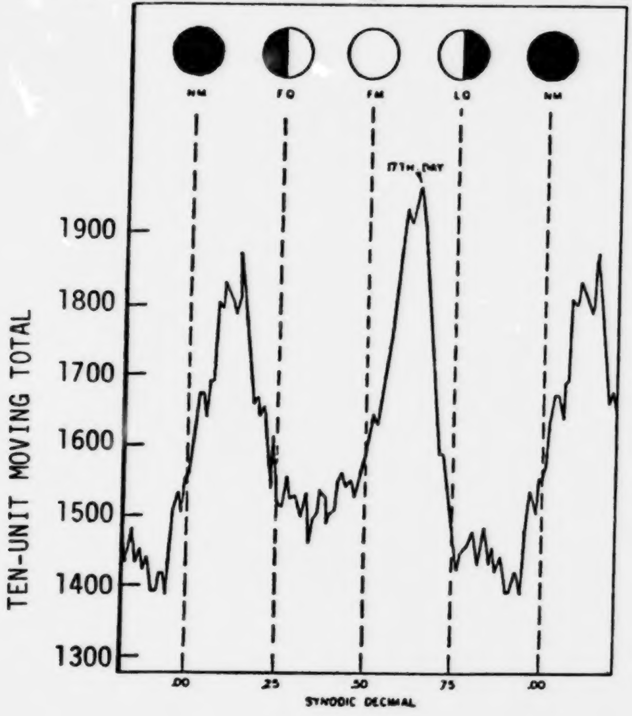

Figure 5: Distribution of the synodic decimals for 16,057 record dates of maximum 24 hour precipitation at 1544 U.S. stations, 1900-1949. The ordinate represents frequencies in terms of 10 units moving totals. (Reproduced from Brier and Bradley, 1964).

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* Two, because the moon crosses the plane of the ecliptic twice in traversing a synodic cycle around the earth.

One of their conclusions is very significant to prairie hydrometeorologists:

> *"......the geographical division of the data suggests a maximum influence of the synodical cycle in the midwestern or central portions of the United States."*

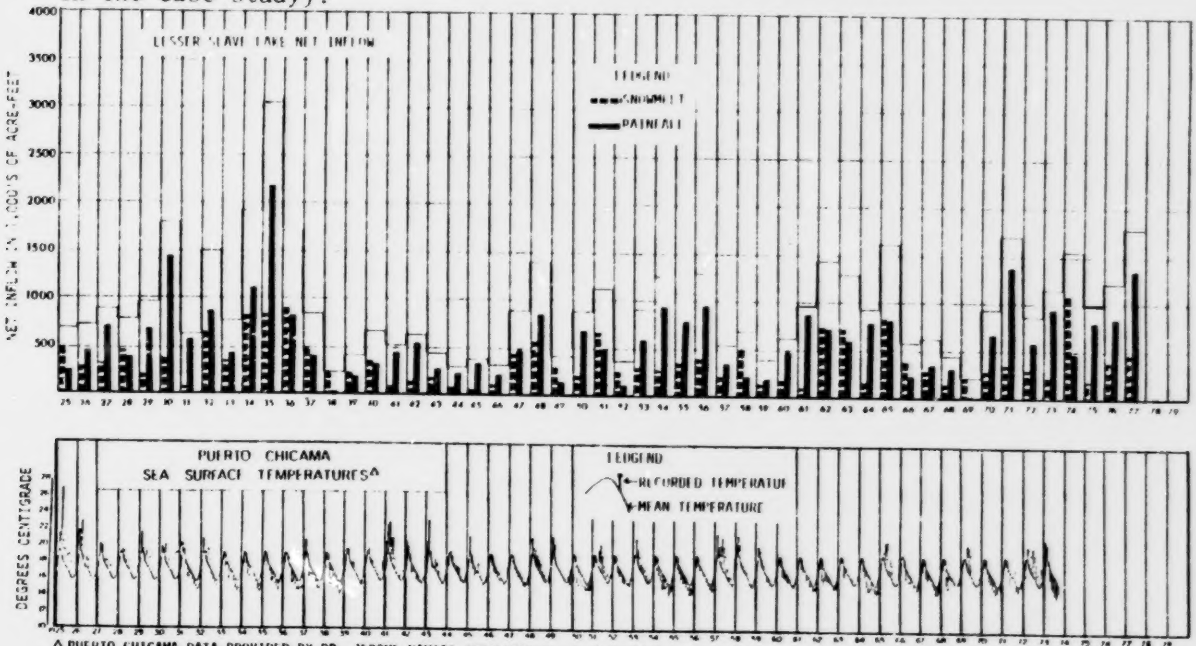
In a later study, Dr. Brier (1968) stated that studies of northern hemisphere weather maps of atmospheric pressure show an oscillation between high and low latitudes with a nodal point aroung 45° north. This oscillation has a very long period as the tide producing force has an 18.6 year period as shown by Smirnov's data (Figure 4).

Pacific Sea Surface Temperatures

The devastating winter of 1976-77 in eastern United States and droughts in the west have been partially attributed to changes in the Pacific ocean temperature regime. A phenomenon called El Nino appears to be responsible. This term is used to describe the appearance of excessibely warm water from coastal Peru westward along the equator dateline. One theory for its origin is provided by Wyrtki (1975). Why an understanding of the El Nino is essential, is best expressed by Dr. Jerome Namias (1975):

> *"El Nino has special appeal to scientists whose primary interests lie in long-term and large-scale phenomena, for EL Nino spans a period of a year and appears to be a manifestation of atmosphere-ocean coupling on an oceanic if not global scale."*

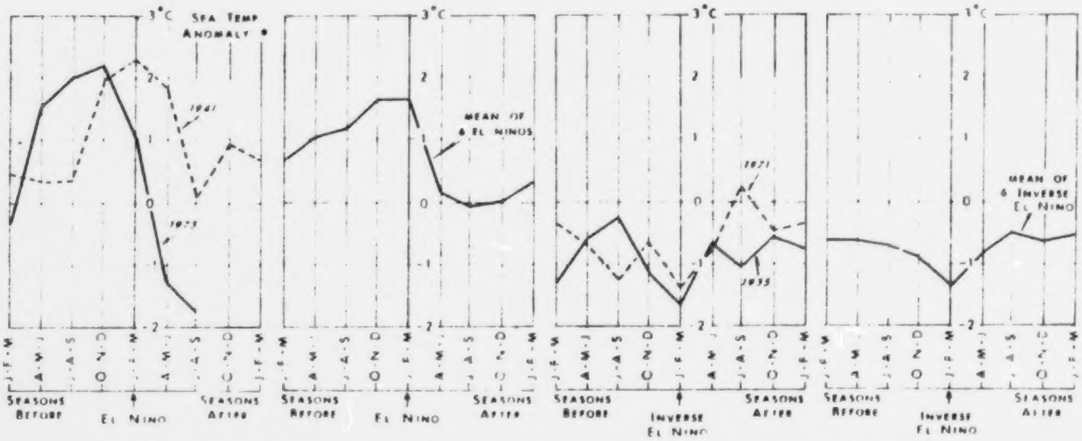
To assist in WSC's drought-flood studies, Dr. Namias provided the plot of monthly sea surface temperatures (SST) reproduced in Figure 6. (The temperature plot is synchronized with Lesser Slave Lake runoffs discussed in the case study).



^ PUERTO CHICAMA DATA PROVIDED BY DR. JEROME NAMIAS (UNIVERSITY OF CALIFORNIA, SCRIPPS INSTITUTE OF OCEANOGRAPHY) AND IS REPRODUCED HERE WITH HIS WRITTEN AUTHORIZATION.

Figure 6: Top: Lesser Slave Lake net inflows.
Bottom: Puerto Chicama Sea Surface Temperatures.

What is significant about the El Nino and Inverse El Nino phenomenon is the persistence (or as Dr. Namias calls it, "coherence") of either above or below normal ocean temperatures. Some of Dr. Namias' plots of SST are presented in Figures 7 and 8 to illustrate the temperature coherence. For example, the 1941 El Nino was heralded by a rise in sea temperatures beginning about August 1940. Maximum temperatures were attained March 1941. Another four months elapsed before temperatures returned to normal.



* SEA TEMP ANOMALY = TEMP DEVIATIONS FROM 49 YEAR MEANS DIVIDED BY STANDARD DEVIATIONS.

Figure 7: Left: Standardized seasonal anomalies of Puerto Chicama sea surface temperatures for seasons preceding and subsequent to the warmest six Southern Hemisphere summers (i.e. J,F,M) Right: Same as at left but for six coldest summers. (Replotted from Namias, 1976).

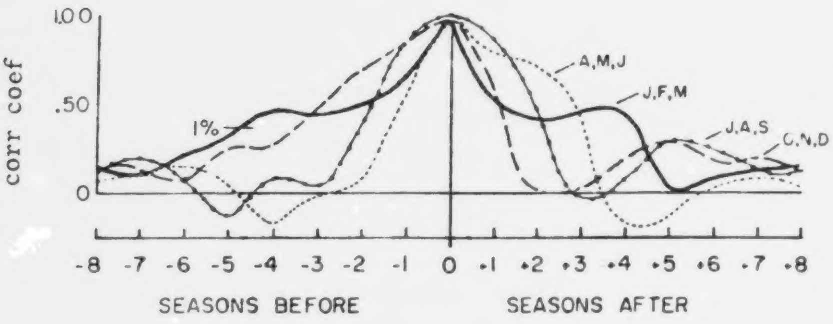


Figure 8: Correlations between average seasonal SST anomalies at Puerto Chicama and antecedent and subsequent seasons. (Reproduced from Namias, 1976).

Three of Namias' observations of the El Nino are relevant to Western Canadian flood studies:

1. It is the central North Pacific which relates to the El Nino, particularly the deep and southward displaced Aleutian low.
2. There is a tendency for El Nino periods to be associated with stronger mid-latitude westerlies.
3. An area of diminished pressure with El Nino occurs at the California and Baja California coasts.

Although WSC's knowledge is still meagre, a brief description of the contribution to our floods of the Inverse El Nino is presented in the case study.

V. CASE STUDY

A case study is presented here to demonstrate how a knowledge of hydrometeorological epochs has value in flood studies and the design of flood control structures. The study region is Lesser Slave Lake (C on Figure 2) in north-central Alberta. In the mid-thirties, when precipitation was deficient over the Great Plains, this region experienced abnormal precipitation, five of six years. More recently, the area has again experienced five floods in six years.

A continuous record of lake levels (from 1914) was compiled from WSC observations or mathematical modelling as reported by Nemanishen and Cheng (1978). To complete the WSC, Lesser Slave Lake Flood Study report (Nemanishen and Meers, 1979) it was found that the existing streamflow records were inadequate to assign a scientifically rational return period for the 1935 flood. Consequently, the SSARR model for Lesser Slave Lake was restructured to calculate all the daily net inflows to the lake from 1916 to 1977.

Using the sea surface temperature data supplied by Dr. J. Namias, correlations were made with the computed Lesser Slave Lake net inflows. These correlations (see Figure 9) suggest the following effects of sea temperatures on runoff in this region:

1. Below normal, October to February temperatures, contributes to high snowmelt runoff as atmospheric circulation tends to deliver heavy winter snowfall in north-central Alberta.
2. Below normal January to June temperatures contribute to higher rainfall runoff as low pressure systems from the Pacific seem to favor a northerly route during May or June.

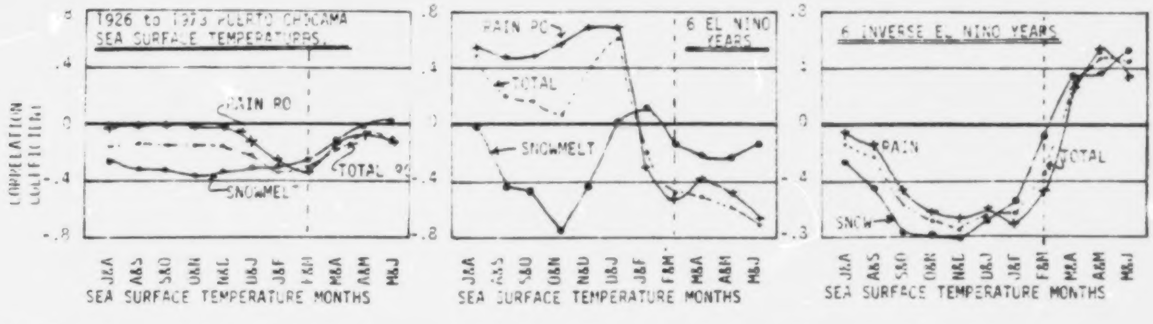


Figure 9: Correlations between Puerto Chicama sea surface temperature and Lesser Slave Lake net inflows.

What about a rational return period for the 1935 flood?

All the available evidence supports the theory that the 1934-35 Lesser Slave Lake flood era was a rare event as the entire hydrometeorological regime of the Northern Hemisphere appears to have been effected:

1. Smirnov (1969) reported that the Volga River regime was greatly reduced due to a persistent change in atmospheric circulation.
2. Tree ring counts by R. Eben-Ebenau provide convincing proof that no greater flood had occurred since at least 1850.
3. The Inverse El Nino phenomenon coincided with the three consecutive flood years of 1934, 1935, and 1936.

In an earlier WSC paper (Nemanishen, 1977) it was recommended that climatological factors be considered in fitting flood frequency curves. On the basis of this knowledge and the evidence that the mid-1930 Lesser Slave Lake flood epoch coincided with a rare but major dislocation of the northern hemisphere climatic regime, the 1935 flood is assigned a return period of at least 1 in 200 years (i.e. 0.5 percent flood). It is highly probable that it may even be a rarer flood like 1 in 500 years as the total rainfall inflow volume of the 1935 flood season was 33 percent greater than in 1920, the second highest rainfall inflow season. On the basis of these observations, a series of flood frequency curves for Lesser Slave Lake is drawn in Figure 10.

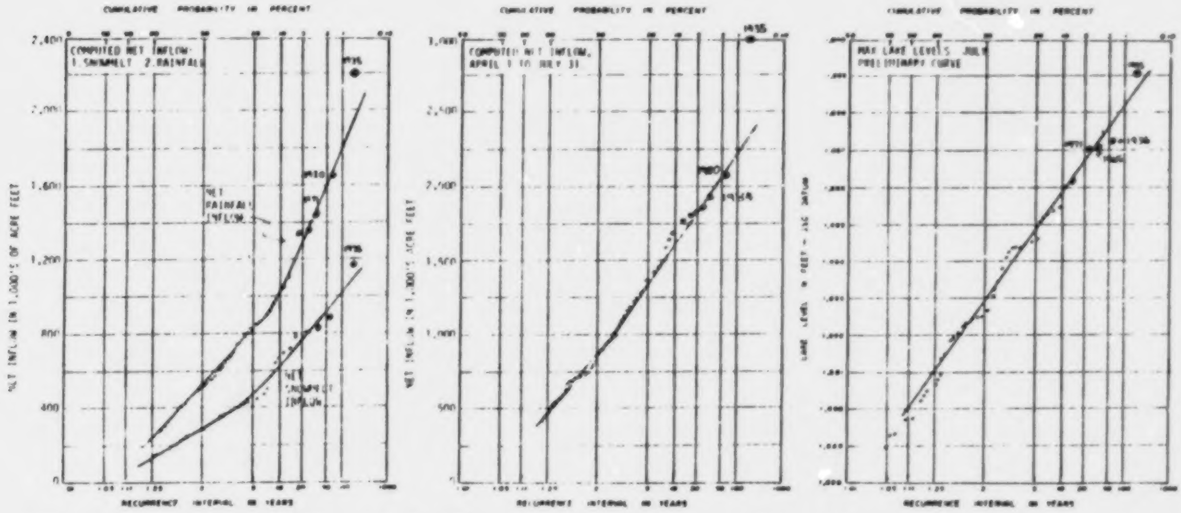


Figure 10: Lesser Slave Lake Net Inflow and Flood Level Frequency Curves.

VI. KNOWLEDGE GAPS AND RESEARCH REQUIREMENTS

Our knowledge of extreme flood events and hydrometeorological epochs - even though very meagre - indicates that as it improves, it will contribute to our ability to predict and effectively control floods. Several knowledge gaps are listed:

1. A workshop in needed to train some IWD engineer - hydrologists in the effective use of band pass filters, spectral and time series mathematical procedures to more productively interpret the basic data now being collected.

2. A limited program should be initiated to cross-correlate runoff from streams, like the Volga, South Saskatchewan and Southwestern U.S.A. Some preliminary WSC studies, using mathematically stacking techniques and data from Marias River indicate that these studies may lead to an improvement in flood prediction procedures.
3. When the new outlet channel for Lesser Slave Lake is excavated this year, the buried vegetative remains from the eastern "ancient" shoreline should be Carbon-14 dated. This would establish the date of the "great" flood which created this shoreline. A tremendous gap in our knowledge of extreme Western Canadian floods would be partially filled by dating.
4. Studies similar to Smith and Younkin's (1972) are needed to determine the role of the polar jet stream on Western Canadian floods.

VIII. ACKNOWLEDGEMENTS

For the opportunity to participate in this Workshop I thank the organizing Committee, G.H. Morton (WSC Regional Chief) and D. A. Davis (Director, Western and Northern Region). The Data provided by Dr. J. Namias and Dr. G. W. Brier was most helpful in the studies. Special thanks is due to Ms. Lillian Brown in drafting the illustrations and typing this paper.

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WORKING GROUP 3

Stochastic Analysis and Operational Research

GROUPE DE TRAVAIL N^{o} 3

Analyse stochastique et recherche opérationnelle

TECHNICAL WORKSHOP

MODELLING ACTIVITIES RELATED TO FLOOD DAMAGE REDUCTION

WORKING GROUP 3-1, EXTREME VALUE AND RISK ANALYSIS

HYDROLOGIC GUIDELINES FOR THE FDR PROGRAM

MAY 23-25, 1979

Robert Condie
Water Planning and Management Branch
Ottawa, Ontario

March 1979

THE THREE PARAMETER LOGNORMAL DISTRIBUTION
APPLIED TO
REGIONAL FLOOD FREQUENCY ANALYSIS BY THE INDEX FLOOD METHOD

Résumé

Regional flood frequency analysis by the Index Flood Method is in fairly common use when estimates of flood flows of specific probabilities are required at ungauged river sites. Use of this method usually assumes that the Gumbel 1 distribution gives a fair representation of the flood frequency régime throughout a homogeneous region, and the most commonly used homogeneity test is based on sampling from that distribution. In Inland Waters Directorate experience, some reservations have arisen regarding the adequacy of the Gumbel 1 distribution, but use of other distributions may have been inhibited by lack of appropriate homogeneity tests. Investigation showed that the three parameter lognormal distribution is generally superior to the Gumbel 1 in Canada. Reasons for this choice are given and the derivation of a homogeneity test is described. Advantages and disadvantages of the Index Flood Method are also discussed in relation to the method of direction regression on the T-year flood.

1. INTRODUCTION

In 1975-76, Inland Waters Directorate in Ottawa was engaged in deriving methodology and preparing guidelines for flood risk mapping in connection with the Flood Damage Reduction Program, and the manual "HYDROLOGIC AND HYDRAULIC PROCEDURES FOR FLOOD PLAIN DELINEATION" was prepared and issued in May 1976. Complementary to the manual, a fully documented computer program FLOOD DAMAGE REDUCTION PROGRAM, FLOOD FREQUENCY ANALYSIS Program FDRPFFA was also prepared in January 1976 with subsequent updates, the latest being March 1979. Use of these two documents generally provides a reasonable estimate of flood frequencies at a gauged site. The manual also gives techniques for flood frequency estimation at ungauged sites including the Index Flood Method, and the manual goes on to state that a probability density function (p.d.f.) representative of the region must be chosen. It follows then that homogeneity testing must be based on sampling from that density function. Up until 1973 regional flood frequency studies in Inland Waters Directorate were made using the Gumbel 1 distribution and the Langbein homogeneity test, and the popularity of this method and distribution was probably due to the readily available documentation by *Dalrymple* [1960]. However, during the development and testing of program FDRPFFA, it was noticed that the Gumbel 1 distribution was not a particularly good fit to the annual flood series from Canadian rivers and an alternative basic distribution was sought.

2. CONSIDERATION OF ALTERNATIVE DISTRIBUTIONS

Program FDRPFFA uses four theoretical distributions, the Gumbel 1, lognormal, 3-parameter lognormal, and log Pearson type III for single station frequency analysis and gives the user guidance in a subjective choice of best fit. These distributions are respectively denoted as G1, LN, 3LN, and LP3. Choice of a best fit is made by comparing unbiassed sample estimates of skewness and kurtosis with their theoretical equivalents and by the appearance of the plot. However it was noted in a test of annual flood series from 37 Canadian rivers that in about 67% of the cases the LP3 was bounded above and that 33% were bounded below. This seemed to violate the concept of homogeneity and the LP3 was not considered further. TABLE 1 gives unbiassed sample estimates of skewness and kurtosis of the untransformed sample, of the transformed sample $ln x$, of the transform $ln(x-a)$, and a comparison with their theoretical equivalents for the 37 stations. For the Canada-New Brunswick flood damage reduction program, Atlantic Region retained a consultant to prepare a regional flood frequency analysis, and in the report *Acres* [1977], using 23 stations in New Brunswick and the adjoining state of Maine, derived the next set of values shown in TABLE 1.

Clearly, of the remaining options, the 3-parameter lognormal distribution is a very sound choice as the basic distribution, if a regional flood frequency analysis is to be prepared in Canada using the Index Flood method. Of interest, but not strictly relevant to Canada, the annual flood series from some 70 rivers in the United States were analysed, and the mean coefficients of skewness and kurtosis of the $ln(x-a)$ series were -0.04 and 3.86 respectively, a good indication of the flexibility of the 3-parameter lognormal distribution.

3. THE 3-PARAMETER LOGNORMAL DISTRIBUTION

There is no need for a detailed description of the distribution here, since it is fully treated in many texts, for example Aitchison and Brown [1957]. Only the probability density function is required for what follows and the derivation of the homogeneity test. If $y = ln(x-a)$ is normally distributed with mean μ and standard deviation σ, then by a change of variate technique the probability density function of x is

$$\phi(x) = \frac{1}{(x-a)\sigma\sqrt{2\pi}} \exp.- \frac{[ln(x-a)-\mu]^2}{2\sigma^2} \qquad (1)$$

The cumulative probability function cannot be expressed in closed form and the general frequency equation is

$$x_t = a + \exp(\mu+\sigma t_p) \qquad (2)$$

where t_p is the standard normal deviate at the required probability level.

3.1 Fitting methods

The most commonly used fitting method is by moments but maximum likelihood, although more complex was found to be better. For the 37 test stations both methods were used and compared and it was obvious that maximum likelihood gave the better fit as can be seen from TABLE 2.

A third fitting method must be mentioned to deal with the special case that arises when the skewness of the untransformed sample is is negative. It can be shown that for negative skew the probability density function at (1) must be rewritten as bounded above. This does not seem compatible with the idea of homogeneity and the fitting method described by *Sangal* and *Biswas* [1970] can be used. Although some distortion will occur, this fitting method can be used for negatively skewed samples and still have the distribution unbounded above.

4. REGIONALIZATION

Most of the work which follows has been programmed and is based on a computer program ML3LN, which fits the 3LN distribution to the sample using maximum likelihood or, if the sample is negatively skewed, by the method of Sangal and Biswas, op.cit.

4.1 Single Station Frequency Analysis

After having estimated the distribution parameters floods are then estimated using Equation (2) and computed for exceedance probabilities of 0·02 to 0·98 by increments of 0·02. These floods are then reduced to dimensionless form by dividing by the flood whose probability of exceedance is 0·50, that is to say the 2-year flood has been chosen as the index flood.

4.2 The Preliminary Regional Dimensionless Curve

The assumption is that the flood frequency regime at an individual station is merely a random sample from the homogeneous region and that a better estimate of the regional characteristics can be obtained by averaging the samples from all stations in the region. Simple arithmetic averaging is used. For each of the aforementioned probabilities the corresponding dimensionless floods at all stations are averaged, thus providing a sample from a Three Parameter Lognormal distribution. The parameters of this distribution are then estimated and are assumed to be the regional dimensionless parameters of a 3LN distribution, unless one or more of the stations in the region is later shown to be non-homogeneous.

4.3 The Homogeneity Test

The test method and criterion are the same as used by Dalrymple. For instance to test any particular station for homogeneity, the estimated 2-year flood is multiplied by the regional Q_{10}/Q_2 ratio to give an estimate of the 10-year flood. From the station analysis the return period of this flood is extracted, which will generally not be 10 years. However, for a given sample size N (record length) this return period should fall within two standard errors of the return period to be expected in random sampling from the Three Parameter Lognormal distribution.

From the p.d.f. given at (1) and defining the reduced variate by

$$t = [ln(x-a)-\mu]/\sigma$$

then by a simple change of variate technique

$$\phi(t) = \frac{1}{\sqrt{2\pi}} \exp. - \frac{t^2}{2} \qquad (3)$$

and the p.d.f. at (3) is the normal distribution with zero mean and unit standard deviation, or t is an N(0,1) vairate. From tables of the normal distribution the value of the variate whose probability of exceedance is 0.1 is 1.28, or the 10-year event for an N(0,1) variate is 1.28. For an N(μ,σ) variate the standard error of estimate of the T-year event is

$$\sigma_T = (\sigma/\sqrt{N}) \ (1+t^2/2)^{1/2} \qquad (4)$$

where N is the sample size and t is the standard normal deviate corresponding to the required recurrence interval T.

So for the 10-year event and unit standard deviation, and since $t_{0.1} = 1.28$, then from (4)

$$\sigma_T = (1+1.28^2/2)^{1/2}/\sqrt{N} = 1.35/\sqrt{N}$$

and $2\sigma_T = 2.70/\sqrt{N}$

From the foregoing, upper and lower 95 percent confidence limits can be computed for any sample size N. TABLE 3 illustrates.

From TABLE 3, if N is plotted as the abscissa and T_l and T_u as the ordinates, then a funnel shaped control chart can be constructed with upper and lower 95% confidence limits.

If working in return periods then it is convenient to construct the control chart on arithmetic Powell time scale probability paper since the return periods are already marked off. However, if working in exceedance probabilities it would be more convenient to use arithmetic normal probabilities since the probabilities are marked off.

5. SOME COMMENTS

There are some practical difficulties which may arise when attempting regionalization by the Index Flood method. As can sometimes happen, two or three basins in the middle of the region may show as non-homogeneous with no possibility of sub-dividing the region to obtain two regressions for mean annual flood. This has always been a drawback with the Index Flood method of regionalization. The validity of the Index Flood method can also be questioned on hydrologic grounds since it implies that the T year flood is always a constant multiple of the mean annual flood, regardless of drainage basin size, whereas observation suggests that the ratio of a flood of some specified return period to the mean annual flood tends to decrease as the drainage area increases. With these difficulties and objections in mind, multiple regression techniques should be considered, and perhaps the most useful of these is the stepwise multiple linear regression.

Objections to the Index Flood method have already been made, and some reasons for preferring the Multiple Regression method are now given. For the Index Flood method a regression equation has to be derived anyway for the mean annual flood. Since basin physiographic parameters have already been extracted, why not use these parameters for regressions on floods of any other required return periods? Furthermore, the ratios of the T year flood to mean annual flood are no longer fixed as in the Index Flood method, but will be a function of the basic physiographic or meteorological characteristics.

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TABLE 1 COMPARISON OF DISTRIBUTIONS

	DISTRIBUTION FITTED					
	G1		LN		3LN	
	C.S. x	C.K. x	C.S. *l*nx	C.K. *l*nx	C.S. *l*n(x-*a*)	C.K. *l*n(x-*a*)
Theoretical	1.14	5.40	0.00	3.00	0.00	3.00
Canada Mean of 37 samples	1.00	5.60	-0.14	3.60	0.00	3.31
New Brunswick-Maine Mean of 23 samples	0.91	4.60	-0.11	3.60	-0.04	3.10
United States Mean of 70 samples	1.64	7.79	-0.15	3.90	-0.04	3.86

TABLE 2 COMPARISON OF FITTING METHODS

	FITTING METHOD	
	M.L.	Moms.
Mean C.S.	0.00	0.12
RMS error about 0.0	0.11	0.27
Mean C.K.	3.30	3.33
RMS error about 3.0	0.81	0.85

For 35 samples only.
Two of the original set
had negative skew.

TABLE 3 HOMOGENEITY TEST 3-PARAMETER LOGNORMAL DISTRIBUTION

N (years)	y	$2\sigma_T = \frac{2.70}{\sqrt{N}}$	Lower Limit		Upper Limit	
			$y - 2\sigma_T$	T_1	$y + 2\sigma_T$	T_u
10	1.28	0.85	0.43	3.00	2.13	60.0
15	1.28	0.70	0.58	3.56	1.98	42.0
20	1.28	0.60	0.68	4.03	1.88	33.0
25	1.28	0.54	0.74	4.36	1.82	29.0
30	1.28	0.49	0.79	4.66	1.77	26.0
40	1.28	0.43	0.85	5.06	1.71	23.0
50	1.28	0.38	0.90	5.43	1.66	21.0
60	1.28	0.35	0.93	5.68	1.63	19.0
100	1.28	0.27	1.01	6.40	1.55	16.5

FLOOD FREQUENCIES BY THE KINEMATIC WAVE MODEL

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April 1979

A paper for presentation at the IWD Technical Workshop "Modelling Activities Related to Flood Damage Reduction", to be held May 23 - 25, 1979 in Hull, Quebec.

FLOOD FREQUENCIES BY THE KINEMATIC WAVE MODEL

The Kinematic Wave Model has been applied to flood frequency derivation for ungauged watersheds by Eagleson[2] and Howard and Smith[3]. This flood frequency model was used by Inland Waters Directorate, Pacific and Yukon Region as one of several methods to estimate design floods for the Alaska Highway Pipeline investigations. This paper briefly describes the basic equations forming the model and discusses the way the model was calibrated for the southern Yukon.

Acknowledgement is given to the Department of Indian Affairs and Northern Development, Water Resources, Whitehorse Y.T. (DIAND), who provided the computer program and much of the required data, and to Charles Howard & Assoc. Ltd., who adapted the Kinematic Wave theory to the Canadian situation and who developed the computer program for DIAND.

Description of the Model

The model is based on the premise that probability of occurrence of a precipitation event varies inversely with its duration and intensity, regardless of geographic location. Rates of variation are defined by the average of measured precipitation durations and intensities in the area. The mathematics involved relate the precipitation events to catchment discharges using various catchment and channel parameters, such as drainage areas and slopes. The model was originally derived for rainstorms, however, it has been adapted to also simulate flood events for snowmelt-generated runoff.

The basic equation is:

$$\text{Prob}\left[Q_p \geq Q_o\right] = \int_{i = \frac{Q_o}{645\,AR}}^{\infty} \beta\, e^{-\beta i - \lambda\left(\frac{a}{i^{1/2}} + \frac{b}{i^{1/3}}\right)}\, di$$

(Definitions of symbols are given in sections following)

Much theory and mathematics are involved in its derivation. Basically, however, it computes the probability that a flood exceeding a given level Q_0 will result from any storm. This is done by integrating the probabilities of all storms large enough to produce a flood of Q_0. The complete mathematical procedure is known as the Kinematic Wave Frequency Model.

Solution of the equation is easily handled by numerical methods in an electronic computer. The parameters involved are described in the following sections (a) through (f).

a) Rainfall Duration and Intensity

An important step in this model is determining the probability of various durations and intensities of rainfall. Equations (1) and (2) below are the basis for the probability function of the model. They simply define, for any storm, the probability of a given duration of rainfall and the probability of a given intensity of rainfall. These are combined in the basic equation above, to provide the probability that any given storm will produce a flood equalling or exceeding a given discharge.

The probability that any storm will last a duration "t_{re}" is given by the equation:

$$f_T(t_{re}) = \lambda e^{-\lambda t_{re}} \tag{1}$$

where;

$f_T(t_{re})$	is the probability density function (i.e. probability of occurrence) of a storm duration t_{re}. If a storm occurs, it has a probability $f(t_{re})$ of a duration t_{re}.
λ	is the inverse of the area's average storm duration (1/hrs.)
t_{re}	is the storm's duration (hrs.)
e	is Napiers e (or 2.71828)

Likewise the probability that any storm will have an average intensity of rainfall "i" is given by:

$$f_I(i) = \beta e^{-\beta \bar{i}_e} \tag{2}$$

where;

$f_I(i)$	is the probability that if a storm occurs it will have an average intensity of i.
β	is the inverse of the area's average rainfall intensity (hrs./in.)
$\bar{i}_e$	is the storm's average rainfall intensity (in./hr.)

By combining equations (1) and (2) we can compute the probability that if a storm occurs it will have an average intensity of i and a duration of t_{re}.

b) Rainfall to Discharge

To compute the probabilities of floods, it is necessary to convert rainfall to discharge. This is accomplished by equation (3).

For any known rainfall excess (the amount of rain that runs directly off the basin) it is a simple matter to compute the equivalent basin discharge if we assume that the rain has lasted long enough and was extensive enough for the discharge to have reached a steady state. In this condition runoff from all points in the basin has had time to reach the project site before the rain has stopped. The time required for this is termed the "time of concentration" of the basin or "t_*". Assuming that the storm duration is longer than t_* the rational formula for discharge applies as follows:

$$Q_p = 645\ AR\ \bar{i}_e \qquad (3)$$

where;

Q_p	is the basin discharge (cfs)
A	is the total basin area (sq. mi.)
R	is theoretically the ratio between area contributing to direct runoff and total basin area
i_e	is the rainfall excess (in./hr.); or rainfall available for runoff after seepage, interception, etc.

c) Basin Time-of-Concentration

Equation (3), which converts rainfall to discharge, is applicable only when the storm duration exceeds the time of concentration of the basin (t_*). Therefore the "t" values used in equation (1) must equal or exceed t_*.

The equation for t_* is defined as;

$$t_* = t_c + t_s \qquad (4)$$

where;

t_c	is the time of concentration of the catchment; time required for the overland flow to reach a steady state under a constant rainfall (hrs),
t_s	is the time of concentration of the stream; the time required for the stream discharge to reach a steady state after the basin's input to the stream has reached a steady state (hrs),
t_*	is the basin time of concentration (hrs).

To relate t_* to various parameters of the catchment and river channel, and to rainfall intensities, the values of t_c and t_s are computed as follows:

$$t_c = 0.913978 \left[\frac{N_c A R}{\sqrt{S_c} L_s \bar{i}_e} \right]^{1/2} \tag{5}$$

$$t_s = 0.130104 \left[\frac{P_s N_s^2}{S_s A R \bar{i}_e} \right]^{1/3} \tag{6}$$

where;

$\bar{i}_e$	is the average rainfall intensity during a given storm (in/hr)
N_c	is Manning's n-value for the catchment (normally 0.30)
A	is the total drainage area (sq. mi)
R	is the ratio of area contributing to direct runoff to total area (see equation 3)
S_c	is the average slope of the catchment
L_s	is the stream length (mi)
P_s	is the wetted perimeter (normally substituted by width) of the stream at the project location (ft)
N_s	is Manning's n-value for the stream at the project location
S_s	is the average slope of the stream.

For simplicity these equations are written as;

$$t_c = a\,\bar{i}_e^{\,-1/2} \tag{7}$$

$$t_s = b\,\bar{i}_e^{\,-1/3} \tag{8}$$

where;

"a" and "b" are the appropriate basin and river constants respectively, computed from the basin parameters in equations (5) and (6)

d) The Frequency Equation

By putting together the relationships described in sections (a), (b) and (c) we can formulate an equation relating rainfall probabilities and basin responses for calculation of the probability that any storm will exceed a given flood level.

The equation is:

$$\text{Prob}\left[Q_p \geqslant Q_o\right] = \int_{\bar{i}_e = \frac{Q_p}{645\,AR}}^{\infty} \int_{t_{re} = t_*}^{\infty} f(\bar{i}_e,\, t_{re}) \quad dt_{re}\, d\bar{i}_e \tag{9}$$

where;

$f\,(\bar{i}_e, t_{re})$ is the probability that when a storm occurs, it has an average intensity of $\bar{i}_e$ and a duration of t_{re}.

This equation simply gives the probability that streamflow discharge due to any storm will equal or exceed Q_p. It is the summation of all the joint probabilities of rainfall intensities greater than "$\bar{i}_e$" and duration "t_{re}". It is important to note that the storm intensity and duration in each case is sufficient to cause a flood of Q_p.

The function $f(\bar{i}_e, t_{re})$ is equations (1) and (2) multiplied, giving the probability of both a rainfall intensity $\bar{i}_e$ and a duration t_{re}. In mathematical terms, $f(\bar{i}_e, t_{re})$ is;

$$\text{Prob}\left[i = \bar{i}_e \ \text{ and } t = t_{re}\right] = \lambda\beta e^{-\beta \bar{i}_e - \lambda t_{re}} \tag{10}$$

Replacing $f(\bar{i}_e, t_{re})$ in equation (9) gives the equation:

$$\text{Prob}\left[Q_p \geq Q_o\right] = \int_{\bar{i}_e = \frac{Q_o}{645\ AR}}^{\infty} \int_{t_{re} = t_*}^{\infty} \lambda\beta\, e^{-\beta i_e - \lambda t_{re}}\, dt_{re}\, d\bar{i}_e \tag{11}$$

where;

λ	is the inverse of average storm duration (1/hrs.)
β	is the inverse of average rainfall intensity (hrs./in.)
i_e	is the intensity of rainfall (in./hr.)
t_{re}	is the duration of rainfall (hrs.)
Q_o	is a given flood related to $\bar{i}_e$

Integrating this with respect to "t" (assuming t is constant) then substituting equations (7) and (8) for t_* gives:

$$\text{Prob}\left[Q_p \geqslant Q_o\right] = \int_{\bar{i}_e = \frac{Q_o}{645\,AR}}^{\infty} \beta\, e^{-\beta \bar{i}_e \;-\lambda\left(\frac{a}{\bar{i}_e^{\,1/2}} + \frac{b}{\bar{i}_e^{\,1/3}}\right)}\, d\bar{i}_e \qquad (12)$$

From definitions for equations (7) and (8);

$$a = 0.913978 \left[\frac{N_c AR}{\sqrt{S_c}\, L_s}\right]^{1/2} \qquad (13)$$

$$b = 0.130104 \left[\frac{P_s N_s^{\,2}}{S_s AR}\right]^{1/3} \qquad (14)$$

This is the basic equation of the Kinematic Wave Frequency Model. (see also page 1). The equation is computed using numerical integration, by an electronic computer. To define the flood-frequency curve at a stream, probabilities for several different floods (Q_o) are computed and are converted to return periods (T) by the following equation:

$$T\left[Q_{max} \geqslant Q_o\right] = \frac{1}{\text{Prob}\left[Q_p \geqslant Q_o\right] \theta\, \phi_{12}} \qquad (15)$$

where;

$T\left[Q_{max} \geqslant Q_o\right]$ is the return period for an annual maximum flood equal to or greater than Q_o (yrs),

$\text{Prob}\left[Q_p \geqslant Q_o\right]$ is the probability that any storm will produce a flood equal to or greater than Q_o (from equation 11),

θ is the average number of storms per year,

ϕ_{12} is the ratio of total precipitation to water running off the basin (i.e. precip. minus interception and groundwater recharge)

Equation (15) gives the return period of the flood Q_o in terms of an annual maximum flood. In other words, the flood Q_o will be equalled or exceeded on the average only once every T years.

e) Combined Rain and Snow Frequencies

Equations (12 through 15) have been derived for rainfall events. However, with the appropriate empirical values for the precipitation parameters (λ, β and θ) snowmelt runoff can also be simulated and the return periods for snowmelt-generated floods can be computed likewise.

The flood estimates included in "Hydrologic Reconnaissance of the Alaska Highway Pipeline Route" have been computed by combining the rainfall-generated floods and the snowmelt-generated floods. A flood-frequency curve was first generated for each of the two types of flood events, and subsequently combined using the following equation:

$$T_c = \frac{T_r\, T_s}{T_r + T_s - 1} \qquad (16)$$

where;

T_c is the composite return period for a flood Q_p (yrs),

T_r is the return period for a rainfall generated flood Q_p (yrs),

T_s is the return period for a snowmelt generated flood Q_p (yrs).

This equation is the final step in the model process. The resulting composite flood-frequency curve is very similar to the flood frequency curve produced from streamflow records, particularly at the higher return periods.

Calibration of the Model

Equations (11 through 14) contain twelve parameters defining meteorological, topographic and hydrologic conditions of a basin. These parameters are listed in Table 1 according to the three categories, together with results of sensitivity tests for southern Yukon streams showing variation in discharge with a 100% change in a parameter value.

It is evident from the sensitivity tests that the greatest opportunity for calibration lies in the average storm intensity (meteorologic parameter β) and in the effective drainage area (topographic parameter R). The meteorologic parameter values were provided by the Department of Indian Affairs and Northern Development (DIAND), Water Resources in Whitehorse. It was assumed that these are reasonably accurate and no attempt was made to refine them. However, in the conversion of the parameters from mean daily to hourly values for the purpose of this analysis, there may have resulted an underestimate of the storm intensities and durations.

The ratio of effective drainage area to total area (parameter R) was, therefore, the main calibrating parameter. The procedure was to vary the ratios until a best fit was attained between modelled flood frequency curves and frequency curves computed from gauging station data. One alternative to this approach would be to determine the ratios by hydrograph analysis, but this was not attempted because of shortage of time.

As a second step, frequency curves were generated based on an identical ratio for all streams. The value chosen was 0.25, a value that produced reasonable results for gauged streams. The results of this test were carefully examined before further runs were made. The test showed a need to vary the ratio with drainage area. Also it was obvious that a better fit at gauged basins could be obtained if the ratios were regionalized.

Since all gauged basins in the southern Yukon with sufficient data for calculating frequency curves have drainage areas greater than 200 square miles, it was not possible to develop parameter R versus drainage area realtionships that could be proven valid for the many small basins along the pipeline route. From other correlation studies for the area, however, it has been shown that floods with a 100-year return period have straight line log-log relationships with drainage area, with slopes ranging between 0.82 and 0.96 for this line. Tests of the model using a constant R-value for all basins produced results which plot roughly on a similar relationship, but with a 1.0 slope, indicating very little variation of unit runoff with drainage area. It was felt that this lack of variation of unit runoff was a major fault of the model that must be corrected. Pending further calibration studies, four regional equations were developed to vary the R-value with drainage area so as to force the 100-year floods to fit a 0.8 slope on the log Q versus log A relationship. The equation for each region took the form $R = kA^a$ where k and a were selected to produce the best fit of the Kinematic Wave model results to observed discharges at gauged stations.

The ratio of total runoff to precipitation (parameter ϕ_{12}) is the second most effective hydrologic calibration parameter; it represents the product of two ratios: the ratio of precipitation that reaches the ground to the total precipitation, and the ratio of direct runoff to water input that reaches the ground. The combined ratios (paramater ϕ_{12}) can be determined by matching rainfall records with streamflow data where such information is available. In the Alaska Highway Pipeline studies the values for ϕ_{12} were obtained for six gauged basins in the southern Yukon area from Reference 1. Using these six basins as a base, and with knowledge of the hydrology, topography and geology of the area, estimates of ϕ_{12} were made for each stream along the pipeline route. The values chosen ranged from 0.28 to 0.65.

Relationships discussed above provided results that fitted the gauged basins well. Also, their extrapolation to ungauged basins appear reasonable.

Recommendations for Further Work

The Kinematic Wave frequency model, when used with some hydrologic judgement, appears to be a useful tool for estimating flood-frequencies at ungauged streams. In the pipeline studies, it was used as an empirical relationship and was calibrated directly to the frequency curves at gauged basins. A more correct method of calibration would be as follows:

i) compute values of λ, β and θ from hourly precipitation data if possible,

ii) compute R and ϕ values at gauged basins from hydrograph analyses,

iii) using all available data, including short term records and temporary gauges, check the relationship of discharge to drainage area for small basins. Ensure that this is reflected by the model; adjust R-values to accomplish this if necessary.

This model gives a convenient method for directly estimating flood frequencies at ungauged basins. A fairly large effort in analysis of meteorologic data is required, but once this is done flood frequency curves can be developed quickly, cheaply and directly. The method should be used with caution, however, and it is not recommended for determining design floods without careful comparison with other methods and with all available hydrologic data relevant to the area. The author has found this to be a useful model for determining preliminary estimates of flood values and it is a convenient tool to provide the hydrologist with a feel for the hydrology of an area.

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Table 1 SENSITIVITY OF PARAMETERS

Parameter	% Change in Q_{100} for +100% change in parameter	Parameter Description
a) Meteorological Category		
β	-52%	inverse of average storm intensity (hrs/in)
λ	-13%	inverse of average storm duration (1/hrs)
θ	+11%	no. of storms per year
b) Hydrologic Category		
ϕ	+11%	ratio of total runoff to precipitation input
R	+95%	ratio of effective drainage area to total drainage area
N_c	-2.1%	roughness coefficient for average basin surface
N_s	-2.0%	roughness coefficient of streambed at project site
c) Topographic Category		
A	not tested (see R)	total drainage basin area (sq. mi)
S_c	+1.3%	average slope of catchment
L_s	-1.4%	length of stream (mi)
P_s	-1.0%	width of stream at project site (ft.)
S_s	+0.8%	average stream slope

FLOOD FREQUENCY ANALYSIS
BY POWER TRANSFORMATION

by

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April 1979

A paper for presentation at the IWD Technical Workshop "Modelling Activities Related to Flood Damage Reduction", to be held May 23 - 25, 1979 in Hull, Quebec.

Flood Frequency Analysis by Power Transformation

Introduction Our starting point is a paper by Chander, Spolia and Kumar (1), in which they describe a method of estimating floods of some desired frequency. They fit the normal probability distribution to an annual flood series after having first applied what they call a power transformation. This paper discusses a computer program to implement this power transformation method and presents some results from analysing 35 streams in British Columbia.

The log-normal distribution is often used in flood frequency analysis; if not because it always gives the best fit to the data, then certainly for its ease of application in obtaining both flood estimates and confidence limits for those estimates. As will be seen, the power transformation is but a generalized log transformation. We can apply the power transformation and obtain an improved fit to the data while at the same time retaining the ease of application associated with using the log normal distribution.

A basic property of the normal probability density function is that it is symmetric about its mean. Therefore, in order to properly fit the normal distribution to an annual flood series, that series should either be symmetric or else be made symmetric, by applying a suitable transformation. Thus, when the log-normal distribution is fitted to a flood series, it is intended that the log transformation produce the required symmetric series. We will now see how imperfect the results can be.

Table I shows the moment coefficient of skewness of the annual flood series of 35 streams located on Vancouver Island and on the Queen Charlotte Islands. The streams have not been selected; every stream with over 10 years of flood record has been included. The untransformed flood series of all but 2 of the 35 streams have a positive coefficient of skewness; the mean deviation from zero is 0.85. On the other hand, the log transformed series generally have a coefficient of skewness closer to that of the normal distribution; now the mean deviation from zero is 0.35. Still, many streams have a significantly skewed distribution.

(1) Subhash Chander, S.K. Spolia and Arun Kumar, Flood Frequency Analysis by Power Transformation, Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 104 No. HY11, November 1978.

With the power transformation, to be outlined next, it is possible to produce a series with coefficient of skewness of exactly zero, ie. a series that is symmetrically distributed.

The Power Transformation The power transformation function is:

$$y(a,x) = \frac{x^a - 1}{a} , \quad x > 0$$

where a is the transformation coefficent,
x is the independent variable,
and y is the transformed variable.

From this definition it is not clear what the transformation is at a = 0. We can, of course, apply l'Hospital's rule, or, better yet, since the result is useful for other computations, substitute the following series expansion:

$$x^a = 1 + a \ln x + \frac{1}{2!} a^2 \ln^2 x + \frac{1}{3!} a^3 \ln^3 x + \cdots$$

We then have the following result:

$$y(a,x) = \ln x \left(1 + \frac{1}{2!} a \ln x + \frac{1}{3!} a^2 \ln^2 x + \cdots \right) , \quad x > 0$$

We can use this formula for values of the transformation coefficient, a, close to zero as well as at zero and use the original definition for all other values. The reciprocal, logarithmic, square root and linear transformations will be seen to be special cases of the power transformation, corresponding to values of the transformation coefficient, a , of -1, 0, ½ and 1 respectively.

Figure 1 is a graph of the family of curves for the power transformation function. The horizontal axis represents values of the transformation coefficient, a, ranging from -1 to +1. The vertical axis represents the normalized, transformed variable, y, for values of the independent variable, x, ranging from 1 to 100. It can be seen that as the transformation coefficient decreases, the higher values of the independent variable are increasingly compressed with respect to the lower values. By choosing the right value of the transformation coefficient the higher values of an annual flood series can be compressed (or expanded) to just the right extent that the transformed series becomes symmetrically distributed.

The inverse of the power transformation is easily obtained by rearranging terms:

$$x = (1 + ay)^{1/a} \quad , \quad ay > -1$$

Again it is not clear what the transformation is at a = 0. This time we can rewrite the transformation as

$$x = e^{\frac{1}{a}\ln(1+ay)} \quad , \quad ay > -1$$

and then substitute the following series expansion:

$$\ln(1+x) = 2\left[\frac{x}{2+x} + \frac{1}{3}\left(\frac{x}{2+x}\right)^3 + \frac{1}{5}\left(\frac{x}{2+x}\right)^5 + \cdots \right] \quad , \quad x > -1$$

We then have the following result:

$$x = e^{\frac{2y}{2+ay}\left[1 + \frac{1}{3}\left(\frac{ay}{2+ay}\right)^2 + \frac{1}{5}\left(\frac{ay}{2+ay}\right)^4 + \cdots\right]} \quad , \quad ay > -1$$

We can use this formula for values of the transformation coefficient, a, close to zero as well as at zero and use the first formula for all other values.

Effect of the Power Transformation Now it remains to be shown that in producing a symmetrically distributed series we have not adversely affected the distribution in other respects. For example, a transformation that produced a rectangular distributed series would be unacceptable even though the transformed series would be distributed symmetrically.

Table 2 shows the moment coefficient of kurtosis of the annual flood series of the same 35 streams as in Table I. The untransformed flood series of many of the streams have a coefficient of kurtosis that departs significantly from that of the normal distribution, namely 3. In fact, the mean deviation from a value of 3 is 1.14. The power transformed series, we notice, generally has an improved coefficient of kurtosis; now the mean deviation from a value of 3 is just 0.34. Evidently we have succeeded in transforming the original flood series to a series that is more normally distributed than before, both with respect to symmetry and with respect to the degree of peakedness of the distribution.

Computer Program   After we have decided upon some measure of defining the degree of symmetry of a series we need only apply some root finding algorithm to determine the value of the transformation coefficient which makes the power transformed series symmetrically distributed. Our program, in the form of a FORTRAN IV subroutine, is just that simple. The moment coefficient of skewness is used as the measure of symmetry and our root finding algorithm uses a mixture of linear interpolation, rational interpolation and bisection, and is guaranteed to converge. We mention, parenthetically, that Newton's method was tried for finding the root but apart from being more difficult to implement it gave no better performance than the present algorithm and did not always converge.

Once the value of the transformation coefficient is determined, the transformed variable corresponding to the T year event, $y(T)$, is estimated from the familiar equation:

$$y(T) = \overline{y} + K(T)\sigma$$

where $\overline{y}$ is the mean of the transformed series,
σ is the standard deviation of the transformed series,
and $K(T)$ is the standard normal deviate corresponding to a return period of T years.

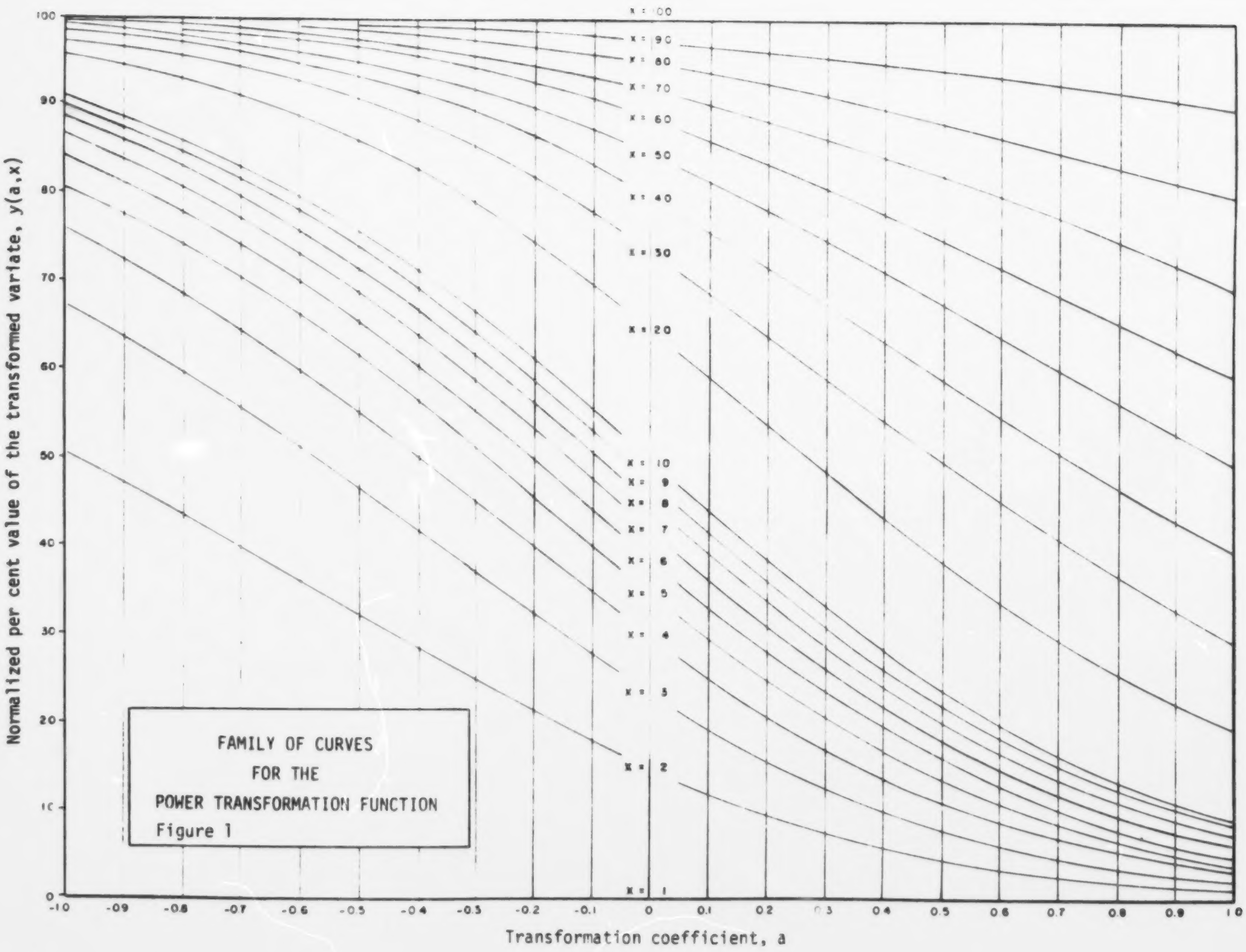
Applying the inverse power transformation to this result gives the flood estimate for the T year return period.

Data input to this subroutine must be handled by the user's calling program. No program listing is given here because some changes are still being tried. However, a program listing, as well as the program itself, will be available at the time of the technical workshop on May 22 - 25.

Results   Table 3 shows the range of values of the transformation coefficient, a, determined for the 35 streams of tables 1 and 2. The values range from just under -1 to just over +1.

The annual peak flows of the 35 streams have been individually plotted on log-extremal probability paper; the plotting positions having been determined by the Weibull formula: T = (N + 1)/M. The flood estimates obtained from using the power transformation method and from fitting the log-Pearson III distribution (by moments) have been drawn on these frequency plots. Table 4 summarizes the differences of the 200 year flood estimates so obtained. In most cases, the two distributions can be seen to be in extremely close agreement. It has been noticed that the 3 largest differences correspond to large negative values of the transformation coefficient. Possibly, roundoff errors in applying the power transformation method can account for these differences. At this writing, there has been insufficient time to investigate further.

An analysis of 35 streams in a hydrogically homogenous area does not entitle us to draw firm conclusions. However, we have found from the 35 stations analysed that the power transformation method gives generally excellent agreement with the established log-Pearson III method and that it is very easy to apply. We hope that readers in other parts of the country will try the program and communicate their findings.



FAMILY OF CURVES
FOR THE
POWER TRANSFORMATION FUNCTION
Figure 1

TABLE 1. COMPARISON OF THE COEFFICIENT OF SKEWNESS - ORIGINAL DATA vs. LOG TRANSFORMED DATA

Station number		Years of record	Skewness Orig.	Skewness Log
08HF003	Kokish R.	27	2.36	1.00
08HD005	Quinsam R.	21	2.07	0.38
08HB022	Nile Cr.	18	1.72	0.38
08HA001	Chemainus R.	24	1.62	0.39
08HB034	Nanaimo R.	42	1.44	0.10
08OB002	Pallant Cr.	11	1.34	0.69
08HD001	Campbell R.	38	1.27	0.51
08HB004	Little Qualicum R.	26	1.24	0.40
08OA002	Yakoun R.	16	1.21	0.40
08HB007	Puntledge R.	39	1.16	0.19
08HB014	Sarita R.	25	1.06	0.33
08HA016	Bings Cr.	14	1.01	0.28
08HC002	Ucona R.	20	0.93	0.10
08HE006	Zeballos R.	17	0.89	0.19
08HA002	Cowichan R.	43	0.86	0.17
08HA011	Cowichan R.	17	0.77	0.12
08HD006	Salmon R.	13	0.77	-0.54
08HC001	Gold R.	20	0.76	0.11
08HA010	San Juan R.	17	0.75	0.18
08HB023	Ash R.	18	0.73	-0.07
08HB008	Sproat R.	57	0.70	-0.19
08HB010	Stamp R.	55	0.63	0.02
08HB024	Tsable R.	18	0.63	-0.20
08HE002	Marble R.	23	0.62	0.04
08HB003	Haslam Cr.	13	0.58	0.03
08HD007	Salmon R.	18	0.46	-0.43
08HB017	Somass R.	17	0.46	-0.22
08HB030	Enos Cr.	16	0.43	-0.32
08HB006	Puntledge R.	20	0.39	-0.95
08HB009	Stamp R.	28	0.31	-0.34
08HB029	Little Qualicum R.	17	0.22	-0.16
08HB011	Tsolum R.	18	0.19	-0.43
08HD003	Campbell R.	22	0.18	-1.28
08HF002	Nimpkish R.	11	-0.04	-0.37
08HA003	Koksilah R.	20	-0.08	-0.69

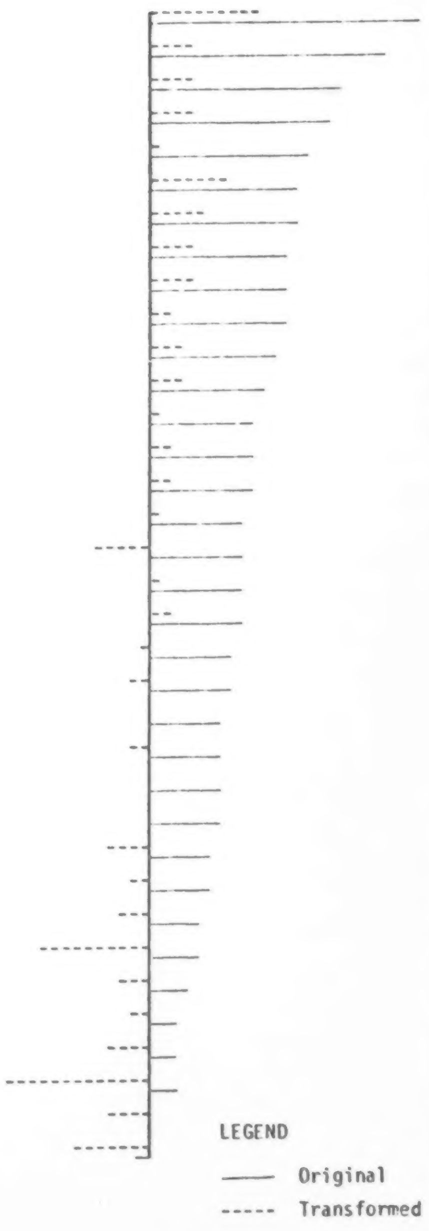


TABLE 2. COMPARISON OF THE COEFFICIENTS OF KURTOSIS - ORIGINAL DATA vs. POWER TRANSFORMED DATA

Station number		Years of record	Kurtosis Orig.	Kurtosis Trans.
08HF003	Kokish R.	27	9.48	3.18
08HD005	Quinsam R.	21	8.99	3.58
08HB022	Nile Cr.	18	7.94	4.20
08HA001	Chemainus R.	24	7.57	4.57
080B002	Pallant Cr.	11	6.13	3.93
08HB034	Nanaimo R.	42	5.85	3.39
08HD006	Salmon R.	13	5.32	5.43
08HD001	Campbell R.	38	4.69	2.57
08HB007	Puntledge R.	39	4.62	3.38
080A002	Yakoun R.	16	4.60	3.48
08HB004	Little Qualicum R.	26	4.45	2.51
08HA016	Bings Cr.	14	4.25	3.44
08HD003	Campbell R.	22	3.88	3.76
08HA011	Cowichan R.	17	3.75	2.81
08HA002	Cowichan R.	43	3.72	2.61
08HB006	Puntledge R.	20	3.59	3.72
08HB014	Sarita R.	25	3.53	2.58
08HB008	Sproat R.	57	3.33	2.85
08HC002	Ucona R.	20	3.25	2.79
08HA010	San Juan R.	17	3.24	3.25
08HE006	Zeballos R.	17	3.20	3.29
08HB030	Enos Cr.	16	3.14	2.93
08HB003	Haslam Cr.	13	3.11	2.93
08HC001	Gold R.	20	3.08	2.30
08HB023	Ash R.	18	2.93	2.47
08HB017	Somass R.	17	2.82	2.83
08HB009	Stamp R.	28	2.81	2.74
08HD007	Salmon R.	18	2.77	2.94
08HB024	Tsable R.	18	2.73	2.88
08HE002	Marble R.	23	2.66	2.61
08HB010	Stamp R.	55	2.63	2.32
08HA003	Koksilah R.	20	2.57	2.50
08HF002	Nimpkish R.	11	2.47	2.46
08HB011	Tsolum	18	2.47	2.50
08HB004	Little Qualicum R.	17	2.02	2.01

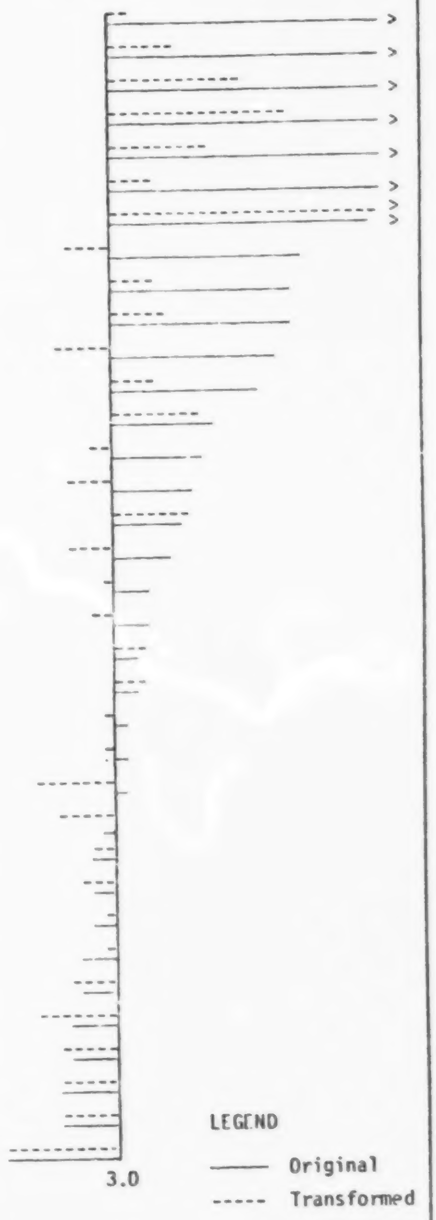


TABLE 3. RANGE OF VALUES OF THE TRANSFORMATION COEFFICIENT

Station number		Transformation coefficient
08HA003	Koksilah R.	1.16
08HF002	Nimpkish R.	1.12
08HD003	Campbell R.	0.87
08HB011	Tsolum R.	0.67
08HB006	Puntledge R.	0.66
08HB009	Stamp R.	0.51
08HB030	Enos Cr.	0.41
08HD007	Salmon R.	0.41
08HB029	Little Qualicum R.	0.38
08HD006	Salmon R.	0.38
08HB017	Somass R.	0.30
08HB008	Sproat R.	0.21
08HB024	Tsable R.	0.19
08HB023	Ash R.	0.08
08HB010	Stamp R.	-0.02
08HB003	Haslam R.	-0.06
08HE002	Marble R.	-0.06
08HB034	Nanaimo R.	-0.07
08HC002	Ucona R.	-0.10
08HC001	Gold R.	-0.15
08HB007	Puntledge R.	-0.19
08HA011	Cowichan R.	-0.20
08HE006	Zeballos R.	-0.20
08HD005	Quinsam R.	-0.24
08HA002	Cowichan R.	-0.26
08HA010	San Juan R.	-0.26
08HB022	Nile Cr.	-0.30
08HA001	Chemainus R.	-0.31
08HB014	Sarita R.	-0.40
080A002	Yakoun R.	-0.42
08HB004	Little Qualicum R.	-0.49
08HA016	Bings Cr.	-0.51
08HD001	Campbell R.	-0.75
08HF003	Kokish R.	-0.79
080B002	Pallant Cr.	-1.06

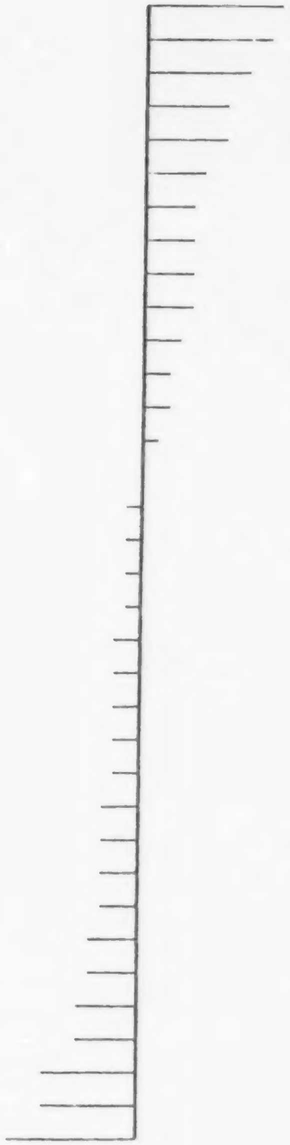


TABLE 4. COMPARISON OF THE 200-YEAR FLOOD ESTIMATES POWER TRANSFORMATION vs. LOG PEARSON III (MOMENTS)

Station number		Power Transformation	Log Pearson III	Per cent Difference
08HB004	L. Qualicum R.	18500	11000	40.5
08HF003	Kokish R.	22300	17000	23.8
08OB002	Pallant R.	5740	4670	18.6
08HA001	Bings Cr.	994	822	17.3
08HB014	Sarita R.	48600	40300	17.1
08HD001	Campbell R.	46800	38800	17.1
08HD003	Campbell R.	30100	27200	9.6
08HB006	Puntledge R.	14000	12800	8.6
08OA002	Yakoun R.	35600	33000	7.3
08HD006	Salmon R.	41000	38600	5.9
08HA010	San Juan R.	51800	49200	5.0
08HC002	Ucona R.	38900	37200	4.4
08HA011	Cowichan R.	28700	27500	4.2
08HE006	Zeballos R.	45300	43500	4.0
08HD005	Quinsam R.	10600	10200	3.8
08HE002	Marble R.	11700	11500	1.7
08HB003	Haslam Cr.	3630	3570	1.7
08HC001	Gold R.	97700	96500	1.2
08HB010	Stamp R.	38200	37800	1.0
08HA002	Cowichan R.	13600	13500	0.7
08HB022	Nile Cr.	1460	1460	0.0
08HB034	Nanaimo R.	51300	51600	- 0.6
08HB008	Sproat R.	14700	14800	- 0.7
08HB007	Puntledge R.	10600	10700	- 0.9
08HB009	Stamp R.	15800	16200	- 2.5
08HD007	Salmon R.	19400	20000	- 3.1
08HB024	Tsable R.	14300	14800	- 3.5
08HB017	Somass R.	55700	58000	- 4.1
08HA001	Chemainus R.	17500	17900	- 4.1
08HA003	Koksilah R.	8050	8390	- 4.2
08HB023	Ash R.	35800	37400	- 4.5
08HB030	Enos Cr.	47	50	- 6.4
08HB011	Tsolum R.	9130	10100	-10.6
08HB029	L. Qualicum R.	8180	9200	-12.5
08HF002	Nimpkish R.	58800	70800	-20.4

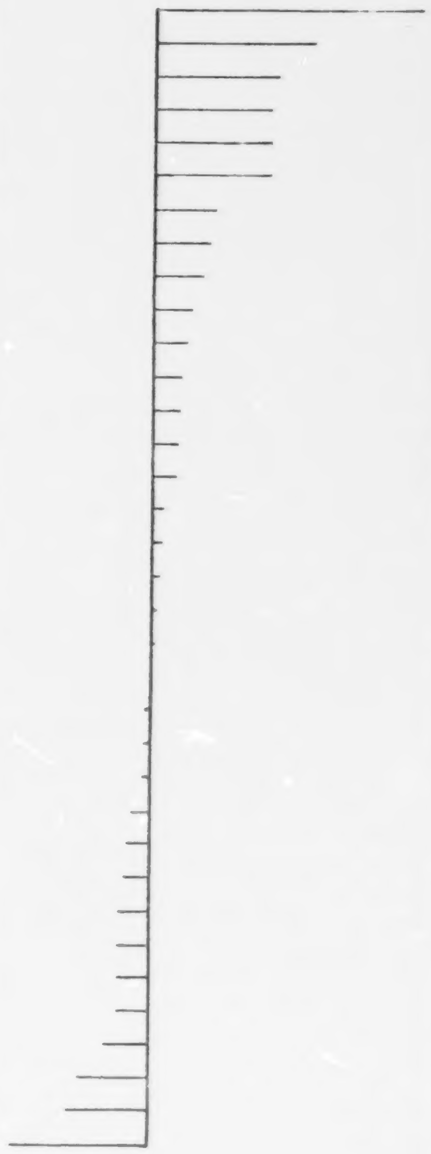
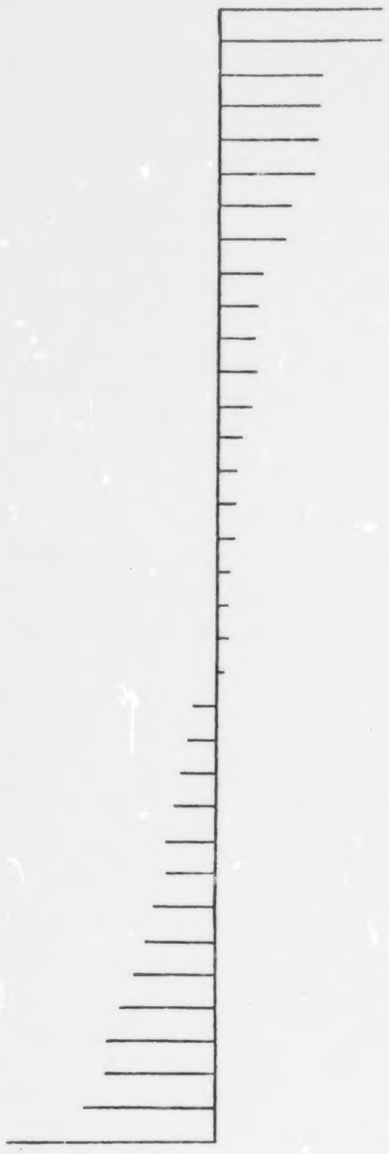
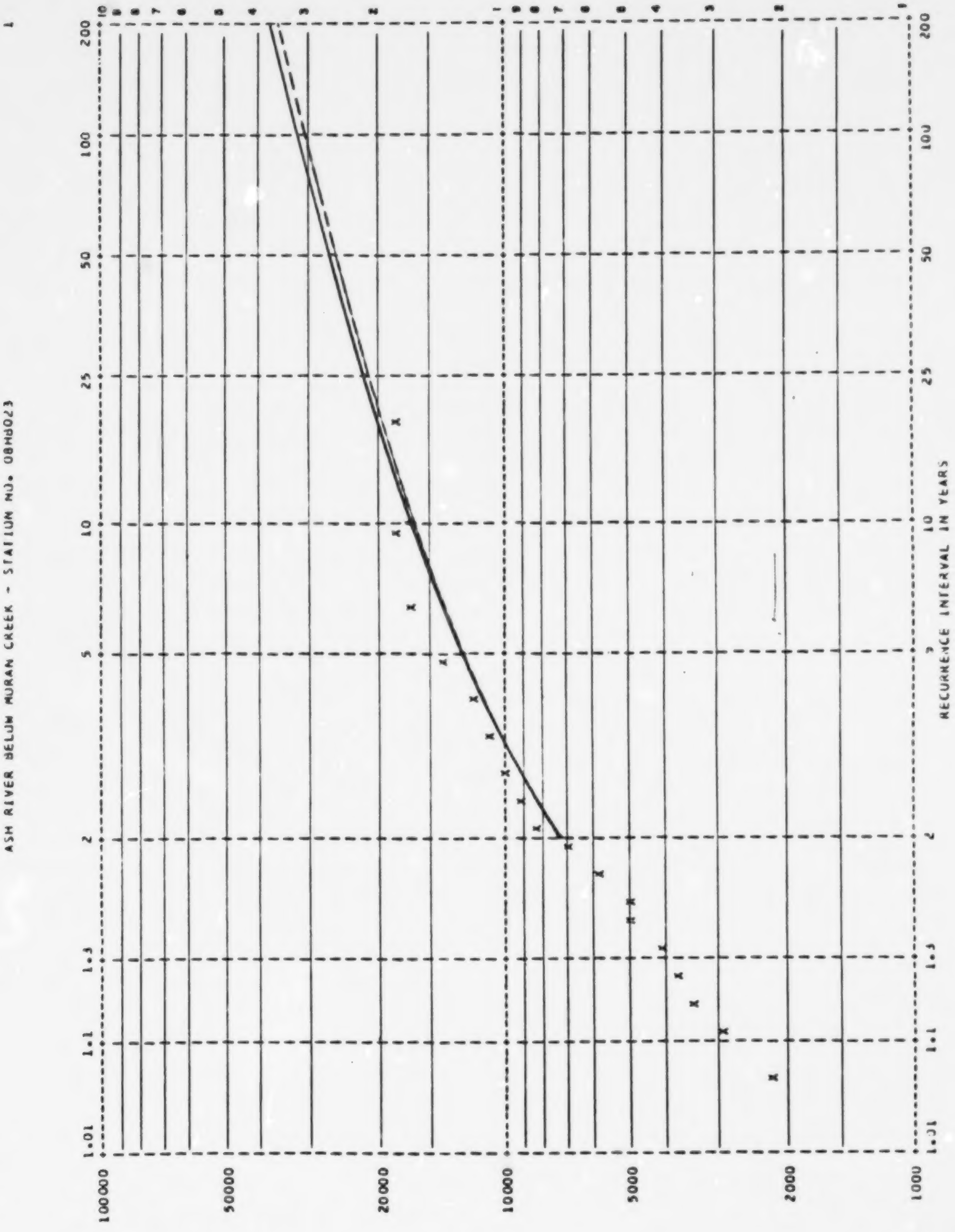


TABLE 5. COMPARISON OF THE 200-YEAR FLOOD ESTIMATES
POWER TRANSFORMATION vs. LOG NORMAL

Station number		Power Transformation	Log Normal	Per cent Difference
08HB004	L. Qualicum R.	18500	8890	52.0
08HF003	Kokish R.	22300	11200	49.8
08OB002	Pallant R.	5740	3860	32.8
08HA016	Bings Cr.	994	699	29.7
08HD001	Campbell R.	46800	33100	29.3
08HB014	Sarita R.	48600	34600	28.8
08OA002	Yakoun R.	35600	28100	21.1
08HD005	Quinsam R.	10600	8510	19.5
08HB022	Nile Cr.	1460	1270	13.0
08HE006	Zeballos R.	45300	40100	11.5
08HA010	San Juan R.	51800	46600	10.0
08HC002	Ucona R.	38900	35000	10.0
08HA011	Cowichan R.	28700	26300	8.4
08HA001	Chemainus R.	17200	16100	6.4
08HB007	Puntledge R.	10600	10000	5.7
08HA002	Cowichan R.	13600	12900	5.1
08HC001	Gold R.	97700	92700	5.1
08HE002	Marble R.	11700	11300	3.4
08HB034	Nanaimo R.	51300	49600	3.3
08HB003	Haslam Cr.	3630	3530	2.8
08HB010	Stamp R.	38200	37600	1.6
08HB008	Sproat R.	14700	15800	- 7.5
08HB023	Ash R.	35800	39100	- 9.2
08HD006	Salmon R.	41000	45500	-11.0
08HB017	Somass R.	55700	63100	-13.3
08HB009	Stamp R.	15800	18100	-14.6
08HB024	Tsable R.	14300	16400	-14.7
08HB030	Enos Cr.	47	56	-19.1
08HB029	L. Qualicum R.	8180	9890	-20.9
08HD007	Salmon R.	19400	24200	-24.7
08HA003	Koksilah R.	8050	10400	-29.2
08HB011	Tsolum R.	9130	12100	-32.5
08HB006	Puntledge R.	14000	18600	-32.9
08HF002	Nimpkish R.	58800	81600	-38.8
08HD003	Campbell R.	30100	48900	-62.5



EXAMPLE PLOT



EXAMPLE TABLE

MAXIMUM DAILY MEAN FLOWS 2

ASH RIVER BELOW MORAN CREEK - STATION NO. 08HB023

DATE	MAXIMUM DAILY FLOW IN CFS
13 DEC 1960	9600
15 JAN 1961	18600
15 DEC 1962	11800
6 FEB 1963	17100
6 DEC 1964	4880
21 OCT 1965	5130
19 DEC 1966	8510
25 DEC 1967	3660
15 JAN 1968	14500
29 MAY 1969	3500
10 APR 1970	2310
10 NOV 1971	9160
18 MAR 1972	7000
16 DEC 1973	5860
16 JAN 1974	10500
4 NOV 1975	18800
27 DEC 1976	2990
14 NOV 1977	4110

RANK	RECURRENCE INTERVAL IN YEARS	MAXIMUM DAILY FLOW IN CFS	YEAR
1	19.0	18800	1975
2	9.5	18600	1961
3	6.3	17100	1963
4	4.75	14500	1968
5	3.80	11800	1962
6	3.17	10500	1974
7	2.71	9600	1960
8	2.38	9160	1971
9	2.11	8510	1966
10	1.90	7000	1972
11	1.73	5860	1973
12	1.58	5130	1965
13	1.46	4880	1964
14	1.36	4110	1977
15	1.27	3660	1967
16	1.188	3500	1969
17	1.118	2990	1976
18	1.056	2310	1970

MEAN ANNUAL FLOOD: 8780 CFS DRAINAGE AREA: 140 SQ MI

STANDARD DEVIATION: 5660 CFS

FLOOD FREQUENCY ANALYSIS
CONCERNS IN THE ATLANTIC REGION

Prepared
for the
IWD Technical Workshop
on
Modelling Activities
Related to the
Flood Damage Reduction Program
Hull, Quebec

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Halifax, Nova Scotia

April, 1979

Introduction

The National Flood Damage Reduction Program is currently being implemented in two of the four Atlantic Provinces: namely, New Brunswick, the first province to sign under the program (March 31, 1976), and Nova Scotia (June 22, 1978). Both provinces have entered into a general agreement and sub-agreements on flood risk mapping and flood studies. In addition, the New Brunswick Government have entered into a flood forecasting agreement (Saint John River) and implementation agreements for flood control in the Marsh Creek Watershed and Southeastern New Brunswick (Petitcodiac River Estuary/Cumberland Basin - Bay of Fundy). This paper focuses on the hydrologic aspects of the flood risk mapping agreements.

The flood risk mapping agreements with New Brunswick and Nova Scotia are virtually identical. In both instances, the flood risk area is defined as that area which would be inundated by the 1:100 year or greater recurrence interval flood while the floodway is defined as that area which would be inundated by the 1:20 year recurrence interval flood. Furthermore, since it is tenuous at best to ascribe probabilities to ice jam floods, a major cause of flooding in Atlantic Canada, the 1:100 and 1:20 year floods are being defined on the basis of open water conditions. In many instances, we will be identifying the extent of a recent large historic flood. Although the historic flood will be shown primarily for information purposes, it may form the basis for designation in certain areas, especially those which have chronic ice jam problems.

The definition of 1:20 and 1:100 year flood flows is a major component of the flood risk mapping program. This paper demonstrates that the accuracy of the flood line is governed primarily by the accuracy of the hydrologic input and that the accuracy of hydraulic modelling and topographic mapping is of secondary importance. In fact, my colleagues within the New Brunswick Government have questioned the need for expensive hydraulic studies and state-of-the-art topographic mapping as envisaged under the National Flood Damage Reduction Program. While I do not condone a relaxation of the national guidelines established for the program, I feel that accuracy required from hydraulic studies and topographic mapping should be evaluated on a site specific basis.

The purpose of this paper is to highlight some of the technical problems we are having in Atlantic Canada in implementing the flood risk mapping program, the approaches we are using or planning to use to overcome these problems and the areas in which we feel more applied research and development and/or further guidelines are required. The discussion is supported by examples from various existing or planned investigations where available.

Single Station Frequency Analysis

Prior to the implementation of the FDR Program in Atlantic Canada, the Gumbel 1 distribution was used exclusively for the purposes of flood frequency analysis. The development and implementation of a documented computer program with attendant plot package has virtually eliminated the tedious task of manual calculation and manipulation of data and, more important, has promoted a uniform approach to flood frequency analysis. Despite the technological improvements which may have been implemented in recent years, the choice of frequency distribution is still, in my opinion, very arbitrary and the errors associated with flood frequency analysis are significant. The following example, taken from recent studies[1,2] undertaken by the Canada-New Brunswick Flood Risk Mapping Technical Committee, supports the above conclusion.

Streamflow records on the Saint John River dating back to 1919 provide 60+ years of good quality record. The hydrometric data recorded on the Saint John River at Pokiok (1919-1967) and below the Mactaquac Dam (1968-present) provided/will provide a significant portion of the hydrologic data base required for the flood risk mapping of the Fredericton and Maugerville/Sheffield/Lincoln flood plains; by far the most significant flood plains, from a flood damage potential perspective, in New Brunswick. In 1973, a flood with a maximum mean daily flow of 11,100 cms (393,000 cfs) at the hydrometric station below Mactaquac caused damages in excess of $6 million in the Fredericton area alone.

Daily Mean Flood Flows: Flood frequency studies of the hydrometric record at the Pokiok gauge and the combined record at the Pokiok/Below Mactaquac gauges have been carried out by various investigators. Estimates of the 1:100 year flood flow from the more prominent studies are summarized in Table 1.

Table 1

Summary of Historical Estimates
1:100 Year Daily Mean Flood Flow
Saint John River Below Mactaquac

Investigation	Period of Record Analyzed	Frequency Distribution Used	1:100 Year Flood Flow (cms)
Saint John River[3] Basin Board (IWD)	1919-1967	Gumbel 1	10,060
N. B. Flood[4] April-May 1973 (IWD)	1919-1973	Gumbel 1	11,300*
N. B. Regional[1] Flood Frequency Analysis (Acres)	1919-1976	3-Parameter Log Normal	9,730

* Extrapolated from Flood Frequency Curve

It is clear from the information for the first two studies that the 1973 flood has had a significant impact on the magnitude of estimates for the 1:100 year flood increasing it by over 12%.

To say that the flood frequency estimate provided by Acres Consulting Services, using the Three Parameter Log-Normal distribution, caused concern to the Canada-New Brunswick Technical Committee is an understatement. After all, the IWD report[4] on the 1973 flood, which received wide distribution, and the pilot flood hazard map[5] prepared for the Fredericton area, which received even wider distribution, both stated that the 1973 flood had a recurrence interval of 84 years compared to a recurrence interval of 500 years as suggested by the Acres investigation. The Committee was not only concerned about the credibility of the flood risk mapping program but with the implication that the 1:100 year flood line would fall well below the 1973 flood line. This prompted a closer look at the methodology used and the results of the Acres investigations. Consultations were held with Headquarters staff[6] of the Water Planning and Management Branch.

The data base, fitted to Gumbel 1 and Three Parameter Log-Normal theoretical distributions, is shown on Figure 1. The sample statistics of the data base together with estimates of the 1:20 and 1:100 year floods are provided on Table 2. The following conclusions were drawn:

1. Goodness of Fit: Because of the small sample size, statistical tests of goodness of fit are seldom effective in hydrology. Nevertheless, it is clear upon examination of Table 2 (standard error) that neither distribution has a significant edge.

STATION 01AK002 SAINT JOHN RIVER AT POKIOK (1919 - 1967)
STATION 01AK004 SAINT JOHN RIVER BELOW MACTAQUAC (1968 - 1976)*

FLOOD FREQUENCY ANALYSIS

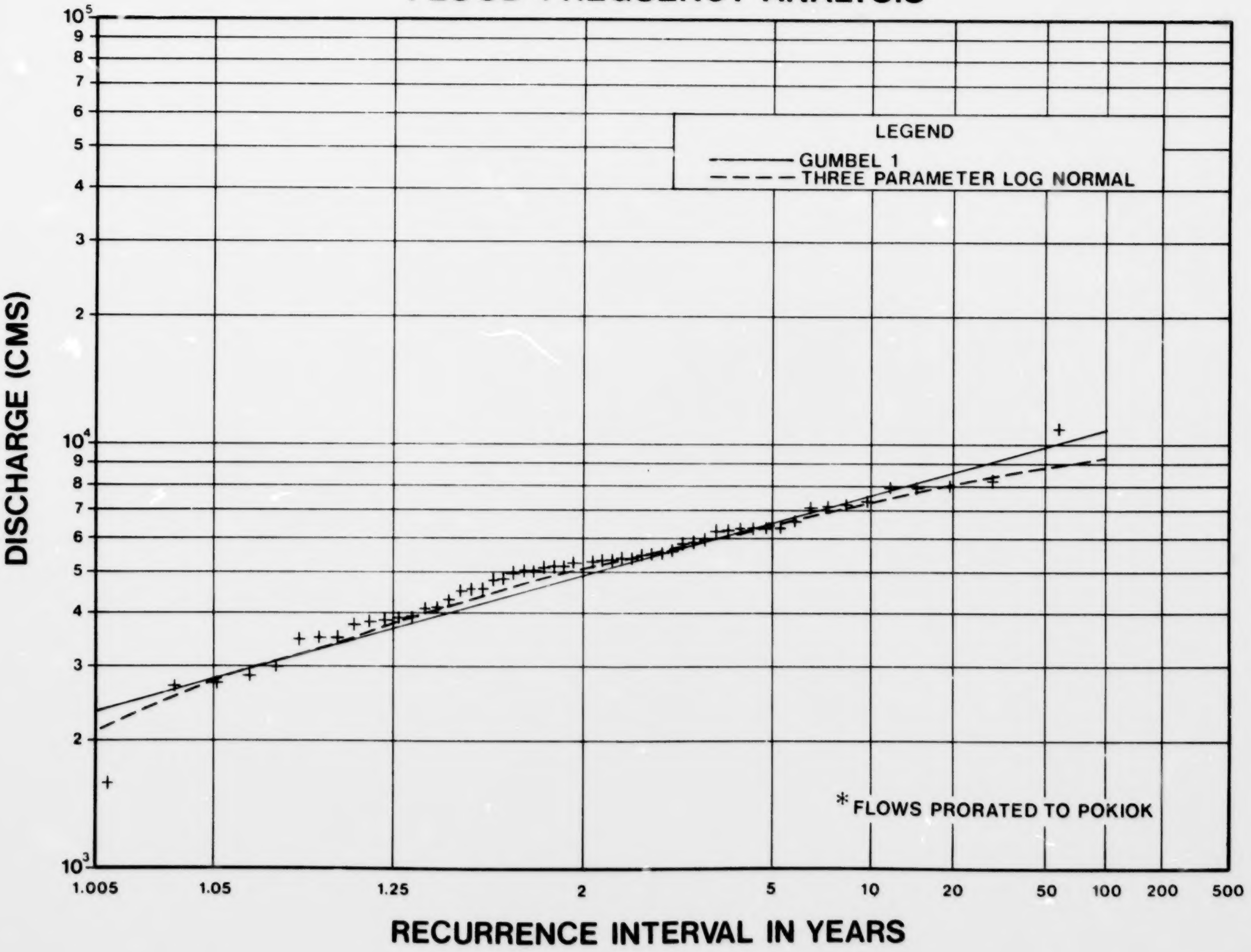


FIGURE 1

Table 2

COMPARISONS OF THREE PARAMETER LOG-NORMAL AND GUMBEL 1 DISTRIBUTIONS

SAINT JOHN RIVER BELOW MACTAQUAC

	Period of Record		Period of Record (Excluding 1965 & 1973)	
	Gumbel 1	3PLN	Gumbel 1	3PLN
Coefficient of Skew	0.545	0.022	0.141	-0.014
Coefficient of Kurtosis	4.54	3.85	2.67	2.65
1:100 Year Flood Flow (cms)	11,600	9,730	10,700	8,950
Standard Error %	6.92	7.62	6.56	6.49
1:20 Year Flood Flow (cms)	9,160	8,270	8,570	7,850
Standard Error %	6.14	5.38	5.72	4.58

2. Coefficient of Skew: Because the coefficient of skew of the sample (0.545) is significantly less than the theoretical coefficient of skew (1.14) application of the Gumbel distribution will overestimate flood flows of longer return periods. On the other hand, the coefficient of skew of the logarithm of the transformed variate (0.022) is very close to the theoretical value of 0.0 for the Three Parameter Log-Normal distribution.
3. Coefficient of Kurtosis: The coefficient of kurtosis of the sample (4.54) is less than the theoretical value for the Gumbel distribution (5.4) which could result in a further overestimation of extreme flows. The coefficient of kurtosis of the logs of the transformed variate for the period of record (3.85) is greater than the theoretical value for the Three Parameter Log-Normal distribution (3.0) which will result in an underestimate of extreme values.
4. Visual Fit: An examination of Figure 1 reveals, to me at least, that the Three Parameter Log-Normal distribution has a superior fit to the sample.

On the basis of the above, I can only conclude that the Three Parameter Log-Normal distribution provides a superior fit of the sample. Consequently, I support the Acres' recommendation. I must admit, however, that the primary basis for my conclusion is the visual plot and not the statistical characteristics of the sample as represented by the 3rd and 4th moments. The fact that the coefficient of skew is close to 0.0, the theoretical value for the Three Parameter Log Normal distribution, is not surprising - after all a transform was applied to the data to remove skewness. The significance of the coefficient of kurtosis is obscure to me - there is little discussion of this characteristic in the literature.

For the sake of completeness, the sample was further analyzed without the 1965 and 1973 floods, the potential low and high outliers respectively. A plot of the data base fitted with Gumbel 1 and Three Parameter Log Normal theoretical distributions is shown on Figure 2. The sample statistics of the data base, together with estimates of the 1:20 and 1:100 year floods, are provided on Table 2. It is clear, based on statistical properties, that the Gumbel 1 distribution does not provide as good a fit as it did for the complete record. However, the primary reason for analyzing the record, less the 1965 and 1973 floods, was to assess the fit that would be provided by the Three Parameter Log-Normal Distribution. As expected, the coefficient of kurtosis was lowered (2.65). However, there does not seem to be a good fit at the higher return periods. I can find no good reason to eliminate the low and high outliers if in fact they are outliers.

A plot of the sample fitted with the Three Parameter Log-Normal distribution together with the upper and lower 95% confidence limits is shown on Figure 3.

The estimates derived for the 1:20 and 1:100 year daily mean flood flows at the hydrometric station below

STATION 01AK002 SAINT JOHN RIVER AT POKIOK (1919 - 1967)
STATION 01AK004 SAINT JOHN RIVER BELOW MACTAQUAC (1968 - 1976)*
FLOOD FREQUENCY ANALYSIS

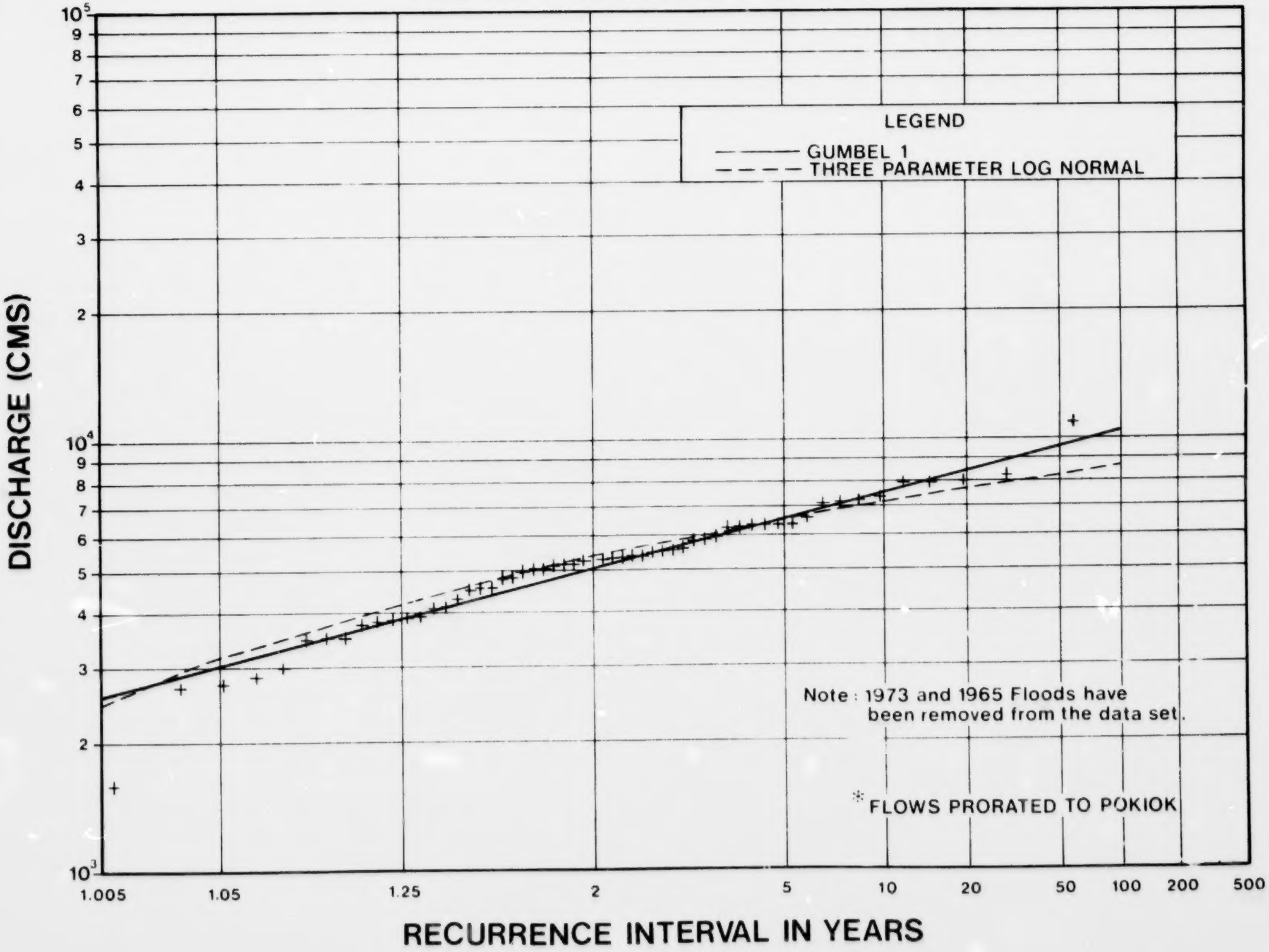


FIGURE 2

STATION 01AK002 SAINT JOHN RIVER AT POKIOK (1919 – 1967)
STATION 01AK004 SAINT JOHN RIVER BELOW MACTAQUAC (1968 – 1976)*
FLOOD FREQUENCY ANALYSIS

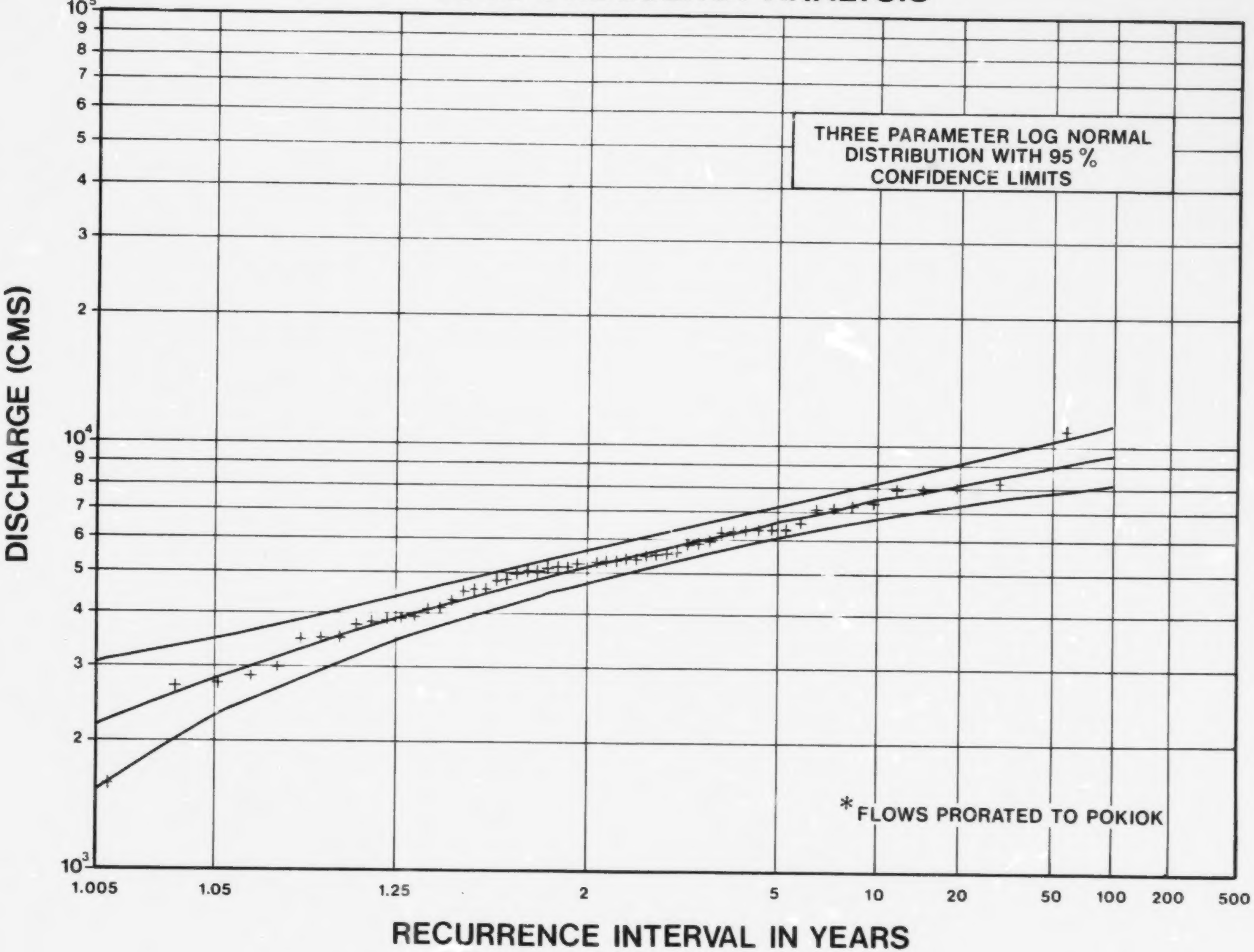


FIGURE 3

Mactaquac are summarized as follows:

	1:20 Year Daily Mean Flood Flow (cms)	1:100 Year Daily Mean Flood Flow (cms)
Upper 95% Confidence Limit	9,260	11,380
Median	8,270	9,730
Lower 95% Confidence Limit	7,360	8,270

Instantaneous Flood Flows: The daily mean maximum flood flow does not provide an adequate estimate of extreme flow in most cases. Even a large river such as the Saint John (drainage area = 39,900 Km^2 at the hydrometric station below Mactaquac), the differences between maximum instantaneous and maximum daily flood flows can be significant. In 1973, for example, the Saint John River peaked at 12,300 cms as compared to the maximum daily mean flow of 11,100 cms, a difference of over 10%. Unfortunately, the instantaneous peak flow data base is rather limited. The Saint John River at Pokiok for example provides only 5 years of record while the gauge below Mactaquac, which was instrumented from the beginning in 1968, has a full record. The record is therefore too short for single station flood frequency analysis. In contrast, two stations on the Saint John River upstream of Pokiok, at Grand Falls and Fort Kent, have over 40 years of instantaneous peak flow records. The Acres study investigated the possibility of developing a relationship between instantaneous peak flows and corresponding maximum daily peak flows. It was recognized at the onset by the Technical Committee that a thorough analysis of flood hydrographs would be required to do justice to this phase of the study. However, the resources required to carry out such a study would be astronomical in view of the fact that archived charts prior to 1970 would have to be analysed. Initially, an equation, similar to the well known Fuller Equation, relating the ratio of instantaneous flood peak to daily mean maximum flow to drainage area was used. Other physiographic parameters, such as basin slope, area of lakes/swamps, when introduced into the equation significantly reduced unexplained variance for all regions except the main stem of the Saint John River. The final relationship developed for the main stem of the Saint John River was as follows:

$$Q_P = 1.037\ Q_D$$

Standard Error of Estimate = 0.013

It was felt that this relationship provided a good fit of the data and was consequently adopted by the Technical Committee.

The 1:20 year and 1:100 year instantaneous flood flows, together with 95% confidence limits, derived for the hydrometric station below Mactaquac, were as follows:

	1:20 Year Instantaneous Flood Flows (cms)	1:100 Year Instantaneous Flood Flows (cms)
Upper 95% Confidence Limit	9,610	11,790
Median	8,580	10,080
Lower 95% Confidence Limit	7,640	8,570

Flood Volume: The lower Saint John River is characterized by large off-channel storage areas. The level to which a given flood will rise is not only a function of its peak flow but also of the state of the system prior to the peak flow event. Therefore, the volume associated with the flood hydrograph must also be considered. At the onset, it was envisaged that a joint probability analysis of flood volume and peak flow would be desirable. However, it was soon realized that it was meaningless to ascribe probabilities to flood volume. Instead, a "typical" flood hydrograph was developed based on an analysis of 20 of the highest floods of record.

Error Analysis: During the conduct of the hydrotechnical study of the lower Saint John River, MacLaren Atlantic Ltd. selected and calibrated the one-dimensional unsteady state flow model, DWOPER, developed by the U. S. National Weather Service for flood forecasting purposes. The model was constructed for the Saint John River from the hydrometric station below Mactaquac to the Reversing Falls located just above Saint John Harbour. The typical flood hydrographs developed for the 1:20 and 1:100 year floods, referred to above, represented the major hydrologic input to the model. Typical flood hydrographs were also developed for the intervening drainage area between the hydrometric station below Mactaquac and the Reversing Falls, using regional flood frequency analysis techniques, and prorated to various locations along the river using the SSARR watershed model.

The following interpretations of error are based on the MacLaren Atlantic Limited report. Because specific simulation runs were not undertaken to determine error associated with hydrologic input at the upstream boundary, the error estimates are not precise but should be within ±0.1 M.

1. Error Associated with Choice of Frequency Distribution:

Should it be shown that the Gumbel 1 distribution best represents the sample, the water level in the Fredericton area would have been underestimated by 0.3 M and 0.7 M for the 1:20 and 1:100 year flood, respectively.

2. Errors Associated with the Three Parameter Log Normal Distribution:

Assuming that the Three Parameter Log Normal Distribution best represents the characteristics of the sample, the error, as represented by the 95% confidence limits, in water levels in the Fredericton area is ±0.4 M and ±0.5 M for the 1:20 and 1:100 year flood, respectively.

3. Errors Associated with Flood Volume:

Errors associated with flood volumes in the Fredericton area are well within ±0.1 M. However, errors of the order of ±0.2 M are possible in the Maugerville area.

4. Errors Associated with Mathematical Modelling:

The DWOPER model was calibrated over three flood events and verified over one. The average accuracy of peak water levels over the study reach was within ±0.2 M.

5. Errors Associated with Topographic Mapping:

The error associated with topographic mapping is ±½ of the contour interval. In the case of 0.5 M contoured maps, the error is ±0.25 M.

The most significant error associated with flood risk mapping in the Fredericton area is obviously associated with the choice of theoretical distribution. However, assuming that the experts in statistical hydrology agree with the choice made, ie. the Three Parameter Log-Normal distribution, it is clear that errors inherent within the estimates produced for floods of long return period are significant and overshadow those associated with hydraulic modelling and topographic mapping. The combined error associated with the Fredericton flood risk maps is ±0.6 M. A similar combined error is inherent in a recently completed hydrotechnical study of the Beechwood headpond on the Saint John River in the Perth-Andover area.

Regional Flood Frequency Analysis

In the case of the Fredericton area, we were extremely fortunate in having a good quality hydrometric station nearby with 60 years of record. In many instances, however, there are little or no hydrometric data available for the stream(s) under consideration. Since the flood risk mapping program in Atlantic Canada is based on probability, the only solution is a regional analysis. This technique provides a mechanism to transfer data from gauged to ungauged sites. The following is a very brief summary of the approach used in New Brunswick together with some typical results.

Frequency Distribution: The Three Parameter Log-Normal Distribution was found to be superior throughout the Province. Where records were too short to properly estimate the skewness of the data, a regional estimate of the coefficient of skew was developed from long term stations and applied to the short term stations resulting in an improved fit of the sample. In some cases, the coefficient of skew was at or close to zero. In such instances, the Three Parameter Log-Normal distribution reverts to the Log-Normal distribution.

Homogeneous Regions: The study identified three homogeneous regions in the Province; the south region which represents coastal drainage to the Bay of Fundy, the main stem of the Saint John River and the North Region which represents the rest of the Province. The regions were defined on the basis of climatic variables such as snowmelt potential and rainfall intensities and on the slopes of individual flood frequency curves. Because of the lack of any meaningfull test for homogeneity, there are probably as many homogeneous regions as there are potential investigators. The choice is very arbitrary.

Basic Regional Analysis Technique: The study employed two basic techniques, the Index Method and the Regression Method. In general, the regression method was found to be superior. The basic regression equation took the following form:

$$Q_T = KA^b \left(\frac{A-A_s}{A}\right)^c FDTS^d$$

Where:

Q_T = Flood flow for T-year return period (cms)

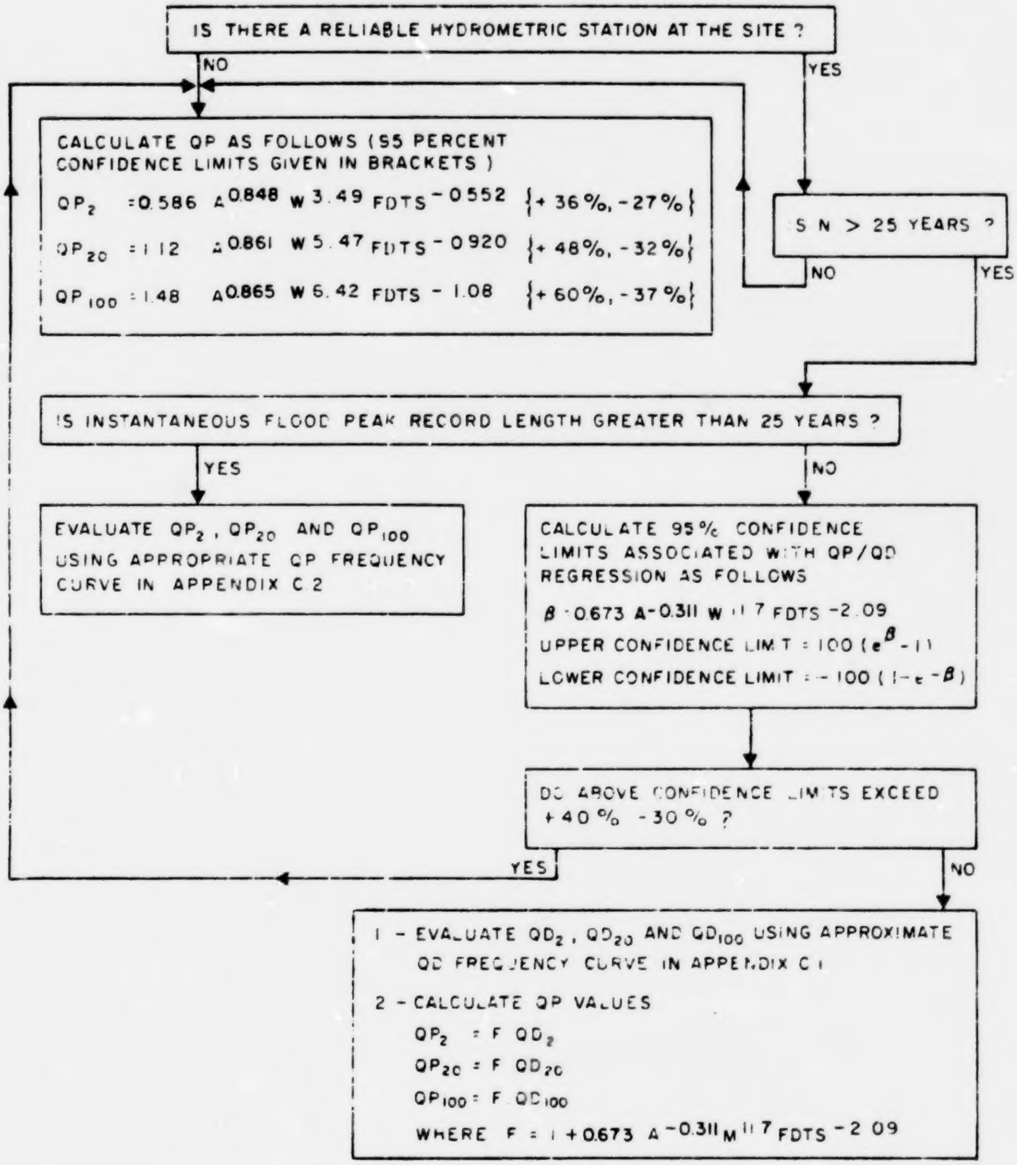
A = Drainage area in Km^2

A_s = Total surface area of lakes and swamps in Km^2

FDTS = Proportion of annual maximum flood peaks occurring in the March, April and May period at the drainage basin centroid

K,b,c,d = Coefficients

Application of Regression Equations: The report presents a recommended procedure to calculate flood peaks, a schematization of which is presented in Figure 4. The procedure is self explanatory so I won't go in to the details. I would like to point out, however, that the constraints to be applied in the application of the regression equation



IS THERE A RELIABLE HYDROMETRIC STATION AT THE SITE ?

CALCULATE QP AS FOLLOWS (95 PERCENT CONFIDENCE LIMITS GIVEN IN BRACKETS)

$QP_2 = 0.586\ A^{0.848}\ W^{3.49}\ FDTS^{-0.552}\quad \{+36\%, -27\%\}$

$QP_{20} = 1.12\ A^{0.861}\ W^{5.47}\ FDTS^{-0.920}\quad \{+48\%, -32\%\}$

$QP_{100} = 1.48\ A^{0.865}\ W^{6.42}\ FDTS^{-1.08}\quad \{+60\%, -37\%\}$

IS N > 25 YEARS ?

IS INSTANTANEOUS FLOOD PEAK RECORD LENGTH GREATER THAN 25 YEARS ?

EVALUATE QP_2, QP_{20} AND QP_{100} USING APPROPRIATE QP FREQUENCY CURVE IN APPENDIX C 2

CALCULATE 95% CONFIDENCE LIMITS ASSOCIATED WITH QP/QD REGRESSION AS FOLLOWS

$\beta = 0.673\ A^{-0.311}\ W^{11.7}\ FDTS^{-2.09}$

UPPER CONFIDENCE LIMIT $= 100\ (e^{\beta} - 1)$

LOWER CONFIDENCE LIMIT $= -100\ (1 - e^{-\beta})$

DO ABOVE CONFIDENCE LIMITS EXCEED +40% -30% ?

1 - EVALUATE QD_2, QD_{20} AND QD_{100} USING APPROXIMATE QD FREQUENCY CURVE IN APPENDIX C 1

2 - CALCULATE QP VALUES

$QP_2 = F\ QD_2$

$QP_{20} = F\ QD_{20}$

$QP_{100} = F\ QD_{100}$

WHERE $F = 1 + 0.673\ A^{-0.311}\ M^{11.7}\ FDTS^{-2.09}$

NOTE

1 - THE ABOVE REGRESSION EQUATIONS ARE VALID ONLY FOR

$25 < A < 5{,}000\ km^2$

$0.85 < W < 1.0$

$0.45 < FDTS < 1.0$

WHERE, $W = \frac{A - A_s}{A}$

2 - REGRESSION EQUATIONS SHOULD BE APPLIED WITH CAUTION TO REGIONS JUST DOWNSTREAM FROM RESERVOIRS IN THE TOBIQUE RIVER SYSTEM - SEE SECTION 5.1 (d)

CANADA – NEW BRUNSWICK FLOOD DAMAGE REDUCTION PROGRAM
REGIONAL FLOOD FREQUENCY ANALYSIS

EVALUATION OF INSTANTANEOUS
FLOOD PEAKS IN THE NORTH REGION

ACRES

FIGURE 4

are based solely on the range of physiographic characteristics present at the gauged sites. This, in my opinion, is a major weakness. An examination of the drainage area, for example, of all hydrometric stations used to develop the relationships will reveal that there is a concentration of stations around the median basin size with only one or two at the extreme ends of the spectrum. Therefore, the application of this particular regression equation for the North Region to a watershed with a drainage area of 25 Km^2 may be tenuous and certainly the error could be more significant than those reported.

Some Typical Results: There are a number of flood plains within the Saint John River Valley which will be mapped. Needless to say, the accuracy of the hydrologic data will be of the same order of magnitude as that presented in the previous chapter. Outside the Saint John River Valley, the Walker Brook flood plain in Campbellton, New Brunswick and the Kennebecasis River/Trout Creek/Parsons Brook/Ward Creek flood plain in Sussex, New Brunswick are ranked high on the priority list for flood risk mapping.

Walker Brook is a relatively small basin in Northern New Brunswick with a drainage area of only 69 Km^2. The estimates provided by the Acres study for the 1:20 and 1:100 year instantaneous flood flow are as follows:

	1:20 Year Instantaneous Flood Flows (cms)	1:100 Year Instantaneous Flood Flows (cms)
Upper 95% Confidence Limit	59	85
Median	40	53
Lower 95% Confidence Limit	27	33

The calculation of flood stage along Walker Brook is complicated by numerous underdesigned highway culverts but otherwise is quite straightforward.

On the other hand, the Sussex flood plain is perhaps the most complex flood plain in New Brunswick because of the lack of data. The merging of streams with differing physiographic characteristics is the primary reason for this. The definition of flood stage will not only depend on estimates of peak flow but on basin lag. The Kennebecasis River at Sussex drains an area of 767 Km^2. The following is a summary of instantaneous peak flows provided by the Acres study:

	1:20 Year Instantaneous Flood Flows (cms)	1:100 Year Instantaneous Flood Flows (cms)
Upper 95% Confidence Limit	1,400	2,200
Median	720	1,100
Lower 95% Confidence Limit	370	550

The Trout Creek watershed drains an area of 223 Km^2. The following is a summary of instantaneous peak flows provided by the Acres study:

	1:20 Year Instantaneous Flood Flows (cms)	1:100 Year Instantaneous Flood Flows (cms)
Upper 95% Confidence Limit	530	820
Median	270	410
Lower 95% Confidence Limit	140	200

It is quite clear from the above that error associated with the definition of flood flows is significant - so much so that it may be meaningless to ascribe probabilities. The errors associated with the 1:20 and 1:100 year flood flows for the Kennebecasis River at Sussex are consistent with those inherent in similar estimates obtained from the hydrometric station on the Kennebecasis River at Apohoqui, downstream of Sussex.

It is admitted that the above examples are extreme. However, the range between 95% confidence limits for extreme flood flows, calculated from hydrometric stations with periods of record in the order of 25-30 years, can be as high as 80%-90%. For some flows in particular areas this may be very significant indeed.

Conclusions and Recommendations

The Three Parameter Log-Normal distribution was selected, over the Gumbel 1 distribution, to fit flood flow data in New Brunswick. Application of the Three Parameter Log-Normal distribution to flood flow data for the lower Saint John River results in a 1:100 year flood profile which is 0.7 M (2.3 feet) lower than that which would be computed using the Gumbel 1 distribution. The credibility of past and current investigations could be questioned when flood hazard maps for the Fredericton area are made public. The choice of which distribution best describes the data base is, in my opinion, based on arbitrary criteria. It is recommended that the technical guidelines established for the flood risk mapping program be reviewed at the national level and updated, if possible, to include greater discussion on the criteria which should be assessed in the selection of the most appropriate flood frequency distribution.

The error inherent in flood frequency estimates exceeds by far the errors associated with the other components of the flood risk mapping program; hydraulic modelling and topographic mapping. The hydrologic data base on the Saint John River is the best, by far, in Atlantic Canada. Despite this, errors in the 1:100 year flood flows, as determined from the 95% confidence limits, translate into an error of ±0.5 M in the flood profile at Fredericton and Perth-Andover compared to hydraulic modelling errors of ±0.2 M and topographic mapping errors of ±0.25 M (0.5 M contoured maps). The combined error inherent in the flood risk maps is ±0.6 M. If the Fredericton area were mapped instead using 1.0 M contour intervals, the combined error would have been ±0.75 M. It is recommended that the accuracy requirements for the hydraulic modelling and topographical mapping components of a flood risk mapping project be defined on a site specific basis taking into consideration errors inherent in the flood flow estimates. Complex and expensive hydraulic studies and detailed topographic mapping may not be warranted in the majority of instances, at least in Atlantic Canada.

The accuracy of flood frequency estimates for flood plains in New Brunswick outside the Saint John River valley degenerates because of the sparceness of hydrometric records. The estimate of the 1:100 year instantaneous flood flow for the Kennebecasis River at Sussex, for example, is 1,100 cms with 95% confidence limits of 550 and 2,200 cms. I can only conclude that it is very tenuous to ascribe probabilities to flood flows at this site and for that matter at many of the flood plains on Schedule "A" to the Canada-New Brunswick Flood Risk Mapping Agreement outside of the Saint John River valley. Rather, I would foster the approach used in much of Ontario, ie. designating flood risk areas on the basis of specified input. I would strongly support the collection of streamflow data at or near the site, as suggested in the technical guidelines, to select, calibrate and verify the appropriate watershed model. While I recognize there are problems with type of approach, I feel that it provides a sounder hydrologic basis for designation purposes.

The Flood Damage Reduction Program is in its infancy. I feel that the policies inherent in the program are sound and am convinced that future generations will be thankful that we had the foresight to implement it. I feel that the program is technically sound for many of our larger river systems where relatively lengthy and good quality hydrologic records exist. I am concerned that we can not meet the technical standards adopted for the program for many of our smaller river systems. I recognize that I am biased in this respect in that many of our rivers in Atlantic Canada are small in relation to the rest of the country. However, should the hydrologic basis for the flood risk mapping program be scrutinized for the Sussex, New Brunswick flood plain, as an example, the credibility of the program could be questioned. I am uncertain as to how the program is being implemented across the country from a technical point of view. A workshop of this nature should begin to clear up some of this uncertainty but it is only the starting point. I recommend that the flow of technical information continue perhaps through a coordinator at the national level. I would further recommend that an ongoing technical audit of the program be implemented not only to ensure that the program is being implemented in a consistent manner across the country but also to develop criteria for the definition of acceptable error in flood lines.

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DESIGN FLOWS
FOR THE
SHAKWAK PROJECT

PREPARED FOR

TECHNICAL WORKSHOP: MODELLING ACTIVITIES
RELATED TO FLOOD DAMAGE REDUCTION
PLACE VINCENT MASSEY,
HULL, QUEBEC
MAY 23-25, 1979

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ABSTRACT

Design flows for the Shakwak Highway Project, the reconstruction of the Haines Road-Alaska Highway, have been estimated by a number of methods. The application of the various methods is outlined and some representative results given.

INTRODUCTION

Upon request of the Department of Public Works, the Inland Waters Directorate undertook a study to provide design flood data for culverts and bridges for the Shakwak Highway Project (Inland Waters Directorate, 1978).

The Shakwak Project is the reconstruction and paving of the Haines Road and Alaska Highway. Figure 1 (Environment Canada, 1978) gives the location map. The project total length, from the British Columbia-Alaska border to Haines Junction, Yukon, and north along the Alaska Highway to the Yukon-Alaska border, is 520 kilometres.

The project is funded by the Government of the United States through the United States Federal Highway Administration. The design and construction will be managed by the Department of Public Works.

Physiography and Climate

The Shakwak Highway Project follows a major geological feature, the Shakwak Trench, for a significant portion of the route. The valley separates the rugged and lofty Kluane Ranges of the St. Elias Mountains on the west from the Ruby Range of the Yukon Plateau on the east. The southern portion of the highway follows another trench-like upland valley, the Duke Depression, and then traverses the Coast Mountains by way of the Chilkat Pass.

Two climatic regions have been identified, a moist maritime climate south of the pass area and the relatively

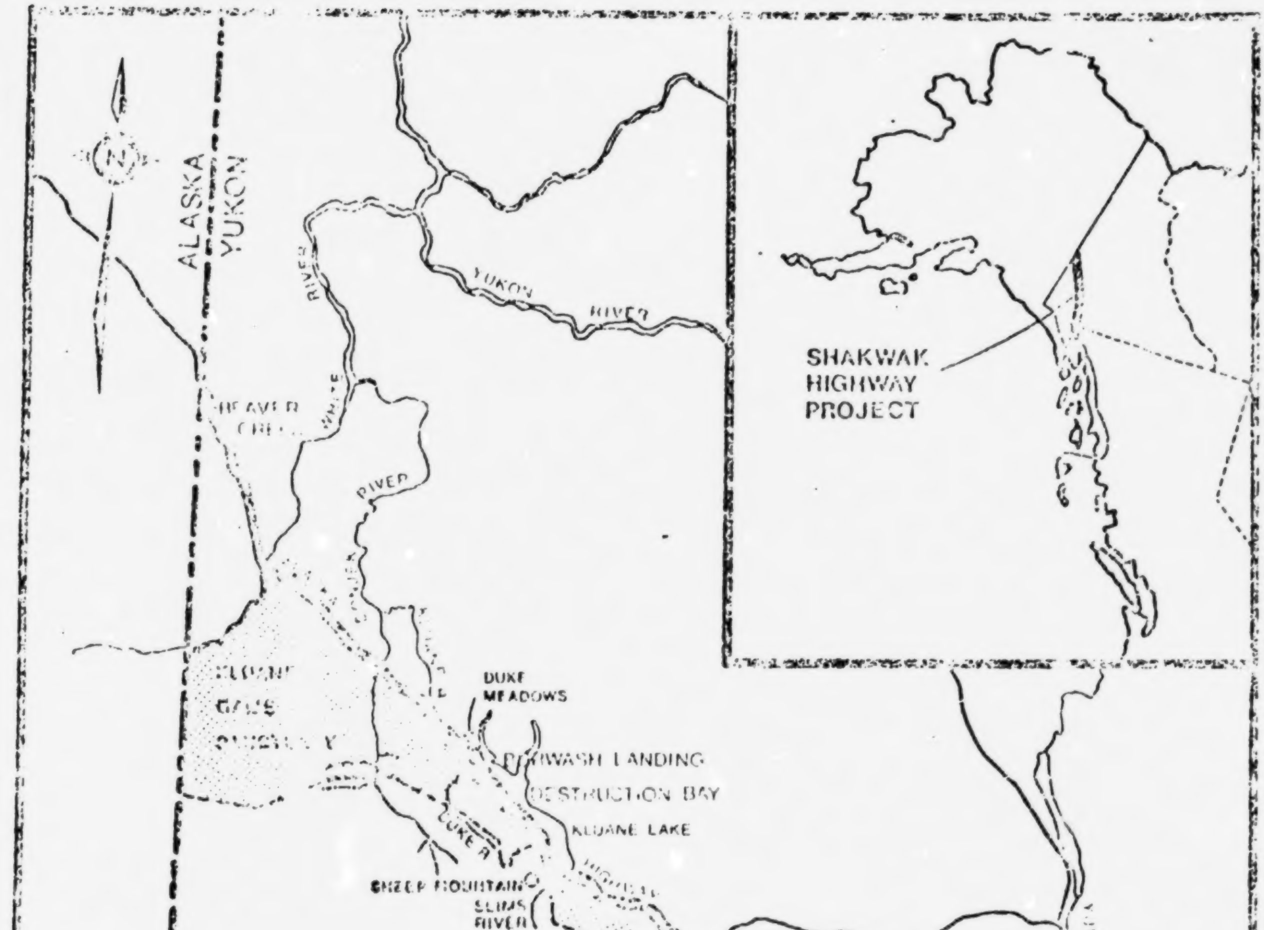
ALASKA
YUKON
YUKON RIVER
WHITE RIVER
BEAVER CREEK
KLUANE GAME SANCTUARY
DUKE MEADOWS
BURWASH LANDING
DESTRUCTION BAY
KLUANE LAKE
SHEEP MOUNTAIN
SLIMS RIVER
HIGHWAY
HAINES JUNCTION
KLUANE NATIONAL PARK
WHITEHORSE
YUKON R.
HAINES
KLUKSHU
CARCROSS
YUKON
BRITISH COLUMBIA
MT. MANSFIELD
GULF OF ALASKA
RAINY HOLLOW
SKAGWAY
HAINES
SCALE/ECHELLE
0 40 80 miles
0 50 100 kilometres
SHAKWAK HIGHWAY PROJECT

FIGURE 1 SHAKWAK HIGHWAY PROJECT

sub-arctic climate of the interior Yukon. Climatological data for Haines APPS No. 2 (Alaska Pipeline Pumping Station), located just north of the B.C.-Alaska border, are representative of the coastal or maritime region. Mean annual precipitation and snowfall are 1272.5 millimetres and 768.9 centimetres (76.89 centimetres water equivalent) respectively. The mean annual temperature is 1.6°C. The Coast and St. Elias Mountains are an effective barrier to maritime influences in the interior Yukon. The mean annual precipitation at Haines Junction decreases to 273.1 millimetres with 134.9 centimetres snowfall. The maximum shadow effect is evident in the Haines Junction area. Northwards, the annual precipitation increases to 443.2 millimetres at Beaver Creek. Annual snowfall is 179.3 centimetres. Mean annual temperatures for the latter two stations are -3.4°C and -6.3°C respectively.

DATA

Climatological Data

An overview of the meteorological factors affecting the Shakwak Project and tables of temperature and precipitation for the climatological stations along the route had been prepared by Wahl (1975). Annual maximum 24-hour rainfall series were obtained for four stations with eight or more years of data. The four stations were Haines Junction, Burwash, Beaver Creek and Snag. In addition, annual maximum 24-hour precipitation data for a number of stations on the mainland of the Alaska Panhandle and the interior Alaska (Tanana Valley) were obtained from the U.S. National Weather Service, NOAA. The data for Haines Terminal, Alaska were considered representative of the coastal region.

Short duration rainfall intensity-duration-frequency relationships were available for only one of the climatological stations along the project route. Short duration rainfall data for Haines Junction based on 10 years of data were prepared by the Atmospheric Environment Service (AES). Short duration rainfall data were also collected for a number of northern B.C. coastal stations for comparison of the coastal rainfall data.

Hydrometric Data

Streamflow data were available for 35 gauging stations operated by the Water Survey of Canada in the interior Yukon and northern British Columbia. However, only one long-term station gauged a stream crossing the project route. Data were also obtained from the U.S. Geological Survey (USGS) for 18 streams on the mainland of the Panhandle and for 49 streams in the interior of Alaska in the Tanana-Yukon River valleys. The USGS has installed many peak stage recorders on small streams and maximum yearly instantaneous peak flow data were available for a range of drainage areas to as small as one square mile.

The Water Resources Section (1975) of the Department of Indian and Northern Affairs commenced in 1975 the Small Stream Investigations Program. Recording gauges were installed on a number of streams throughout the Yukon. Presently, six gauging stations are in operation on streams that cross the highway and have one to three years of data. Unfortunately, no significant runoff events have occurred during the period of operation of these gauges.

METHODS OF ANALYSIS

Rainfall-Runoff Methods

The majority of crossings along the Shakwak Project are small streams for which data were very limited. Two methods of rainfall-runoff analysis, the Soil Conservation Service (SCS) triangular unit hydrograph and Rational formula methods, were applied for estimation of design flows for small drainage areas.

For the application of these methods, design rainfall amounts are required for various durations and return periods. The estimation of short duration rainfall will be summarized briefly in this section. Most of the existing precipitation data were for 24-hour durations. Relationships of short duration to maximum annual 24-hour rainfall were determined using the AES short duration analyses for the few recording stations in the study region and north coastal B.C. Amounts for various durations, i.e. 15 minutes, 30 minutes, 60 minutes, etc., were divided by the 24-hour amount to obtain a set of ratios. Ratios were calculated for the 25-year, 50-year and 100-year return periods and the average determined. The average set of ratios was plotted on semi-logarithmic paper and a smooth curve drawn. Ratios of short duration to 24-hour rainfall are plotted in Figure 2 for some selected stations.

Regional design storms of various duration could then be estimated by multiplication by the ratio for the required duration times the t-year 24-hour rainfall which was established by frequency analysis of the 24-hour data. Appropriate runoff conditions were assumed for calculation of discharges by the SCS and the Rational formula methods. Design flows were estimated for representative streams in each region for small drainage areas up to 25 square miles. Results for the 50-year return period are shown in Figures 3 and 4 for the coastal and interior regions. The maximum recorded floods for the coastal and interior Alaska data are also shown. The results from the SCS and Rational formula methods were considered conservative and not recommended for design purposes.

Regional Flood Frequency Analysis

Two methods of regional flood frequency analysis were attempted. The first method involved the development of regression relationships between various t-year events and the parameters

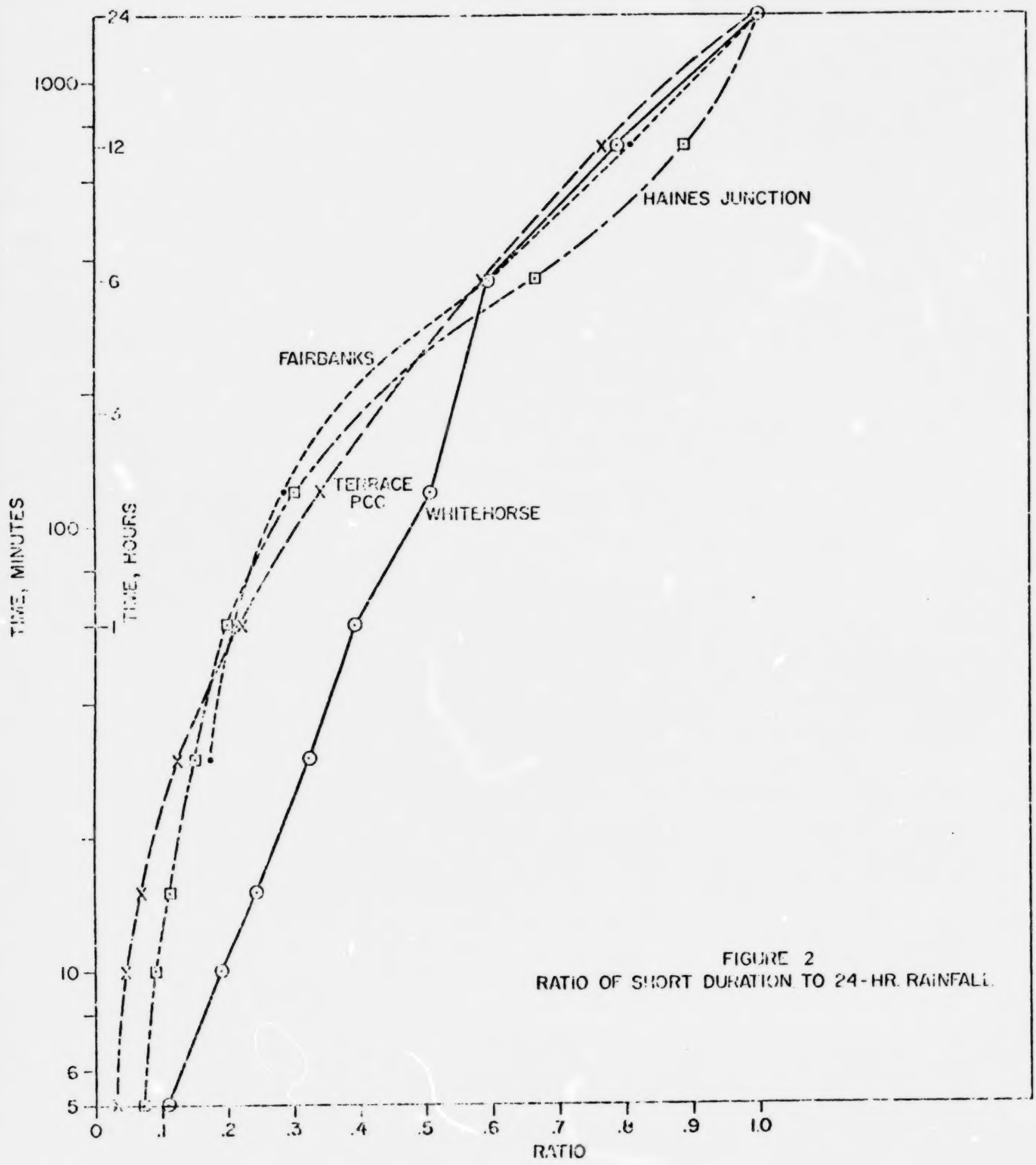

FIGURE 2
RATIO OF SHORT DURATION TO 24-HR. RAINFALL

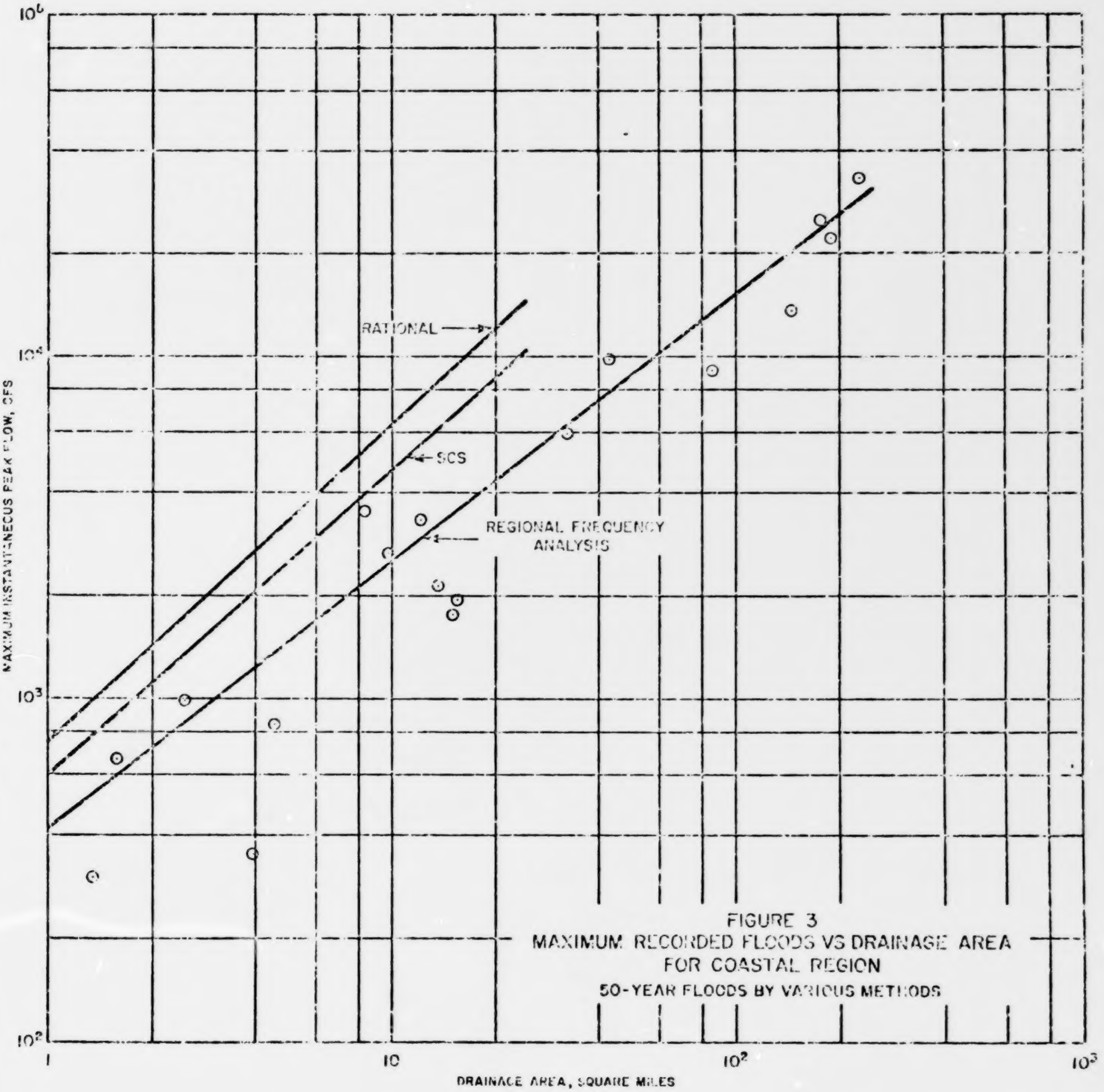

FIGURE 3
MAXIMUM RECORDED FLOODS VS DRAINAGE AREA FOR COASTAL REGION
50-YEAR FLOODS BY VARIOUS METHODS

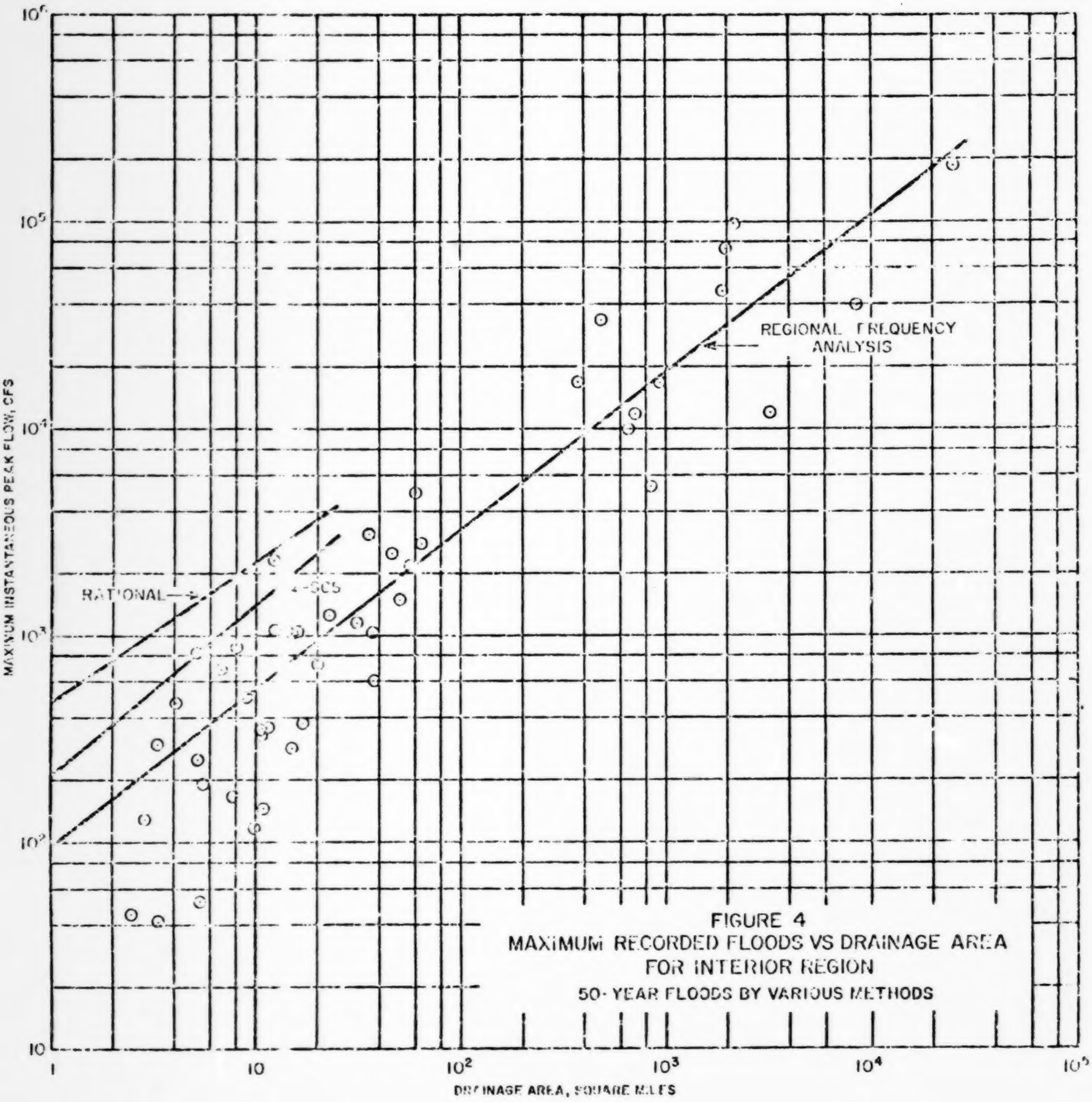

FIGURE 4
MAXIMUM RECORDED FLOODS VS DRAINAGE AREA
FOR INTERIOR REGION
50-YEAR FLOODS BY VARIOUS METHODS

obtained from the square grid data bank. For the Yukon stations with 10 or more years of data, frequency analysis were performed with the use of program FDRPFFA (Condie, Nix and Boone, 1976). Although the results by the regression method were very good as determined by the usual statistical measures, the method was not used for a number of reasons. The Alaskan portion of drainage basins and corresponding parameters had not been included in the square grid data bank. The average elevation parameter for a number of streams originating in the St. Elias Mountains was much greater than the values used in the development of the regression equation. Finally, small basins could not be delineated accurately on the 10 x 10 kilometre grid.

The index flood method was applied using the Alaska coastal and interior peak flow data to delineate two regions. A similar approach had been used in a regional flood frequency analysis for southern Ontario (Sangal and Kallio, 1977). It was felt that the Alaska data from the Tanana-Yukon River valleys would be more representative of conditions along the Shakwak Valley than the WSC Yukon stations which gauge streams on the interior Yukon Plateau. Instantaneous peak flow data were also available for small drainage areas.

The application of the index-flood method has been described in previous publications and will not be repeated here. However, a few salient points for this study are noted. Flood frequency estimates for the Alaska peak flow data were computed using computer program FDRPFFA. The frequency estimates from the distribution judged to give the best fit were divided by the arithmetic mean annual flood to yield the set of dimensionless ratios. Average ratios were determined for each region. The regional flood frequency lines giving the recommended peak flow estimates are also shown in Figures 3 and 4. These results compared well with a study performed for Alaska (Berwick, Childers and Keuntzel, 1964).

DISCUSSION

In the application of rainfall-runoff methods, the major problems were concerned with the magnitude of the design storm and how it is transformed by the basin into runoff. However, the required runoff hydrographs and associated rainfall data were not available to determine the hydrograph parameters or response times for small streams in the Yukon. With regard to precipitation data, the present climatological stations in the Yukon are located in the valleys and no estimation of the increase of precipitation with elevation was possible. Also, the estimation of design rainfall would yield more representative values if data were collected and published on the timing, amounts and duration of individual rainfall events.

Of all the methods applied, the regional flood frequency results were considered the most applicable. The transposition of Alaska data to the Shakwak area was considered appropriate because the two regions are similar in climate and physiography. The results of the regional flood frequency analysis can be checked as more data become available from the DINA Small Stream Program.

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A FLEXIBLE FAMILY OF STOCHASTIC MODELS

by

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ABSTRACT

The family of Box-Jenkins models and other closely related stochastic models, form a group of models which can be used to help solve many types of problems which arise in hydrology. A model which is of particular importance in hydrology is the intervention model. This model can be employed to determine the effects of man-induced or natural interventions upon the mean level of a time series, to fill in missing observations in a time series and to answer other kinds of similar problems. Following a description of the intervention model, it is explained how the most appropriate type of intervention model can be selected to fit to a specified data set by adhering to the identification, estimation and diagnostic check stages of model development. In a case study it is demonstrated how the intervention model can describe the effects of a forest fire upon the average monthly flows of the Pipers Hole River in Newfoundland. In addition to modelling the initial increase in the mean level of the spring flows just after the fire, the fitted intervention model also accounts for the effects of vegetation regrowth which over the years causes the flows to return to their condition prior to the fire. Other varieties of potential applications of the intervention model, confirm the importance of this model for modelling Canadian hydrological data.

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INTRODUCTION

Intervention analysis [Box and Tiao, 1975] is a stochastic modelling technique to analyze the effects of man-induced and natural interventions upon the environment. Although intervention analysis should prove to be useful in many fields of study, the method is especially suitable for utilization in hydrology and other environmental sciences. For example, Hipel et al. [1975] employed intervention analysis to determine statistically the change in mean level of the average annual flows of the Nile River in Egypt due to the construction of the Aswan dam. The technique was used by Hipel et al. [1977b] to ascertain the stochastic effects of reservoir operation upon the downstream seasonal flows of a large Canadian river. Intervention analysis was employed to determine the effects of a forest fire upon the seasonal flow regime of a river [Hipel, 1975; Hipel et al., 1977c] and this example is also presented later in this paper. The method has also been utilized to determine the effectiveness of of pollution abatement procedures for improving air [Box and Tiao, 1975] and water quality [D'Astous and Hipel, 1979]. Furthermore, intervention analysis was used to estimate missing data points in a time series [D'Astous and Hipel, 1979] and Lettenmaier et al. [1978] have demonstrated how a data collection scheme can be designed based upon the power of the intervention model.

The purpose of this paper is to demonstrate the importance of intervention analysis in hydrology. This is effected by pointing out the flexibility of intervention analysis by means of a brief mathematical description of the model and by explaining how the model can be easily implemented in practice by following the identification, estimation and diagnostic check stages of model construction. The practical example demonstrates how the intervention model can describe the increase in the mean level of the spring flows of the Pipers Hole River in Newfoundland due to a forest fire. Besides modelling the flood potential of the spring flows, the intervention model reflects the effects of vegetation regrowth which over the years causes the monthly flows to return to their condition before the forest fire.

THE GENERAL INTERVENTION MODEL

The mathematical description of the intervention model is briefly outlined in this section while the reader may wish to refer to the papers by Box and Tiao [1975] and Hipel et al. [1977b] for a more detailed account of intervention analysis. When there are I_1 interventions the intervention model for a single time series can be written as

$$y_t - \bar{y} = \sum_{i=1}^{I_1} v_i(B)\xi_{ti} + N_t \qquad (1)$$

where $y_t = F(Y_t)$ is an appropriate Box-Cox transformation [Box and Cox, 1964; Hipel et al., 1977b] of the given Y_t series, $\bar{y}$ is the mean of the entire y_t series, B is the backward difference operator defined by $By_t = y_{t-1}$, $v_i(B)$ is the ith transfer function, ξ_{ti} is the ith intervention series which is assigned values of 1 and 0 according to the occurrence and non-occurrence respectively of the ith intervention, and N_t is the noise term which can be modelled by a Box-Jenkins seasonal model [Box and Jenkins, 1970].

When analyzing environmental time series and also river flow data sets where the measuring gauge may break down or perhaps may become inaccessible during severe climatic conditions, often there may be missing data points. The inherent flexibility of the intervention model allows this model to be used to fill in gaps in the data. Suppose, for example, that there are no external interventions and the time series has one missing data point. For the case where the given series has not been transformed by a Box-Cox transformation, the intervention model for determining an estimate for the missing data point may be written as

$$Y_t - \bar{Y} = \omega_{01}\xi_{t1} + N_t \qquad (2)$$

where ω_{01} is the parameter of the transfer function and the intervention series ξ_{t1} is given a value of unity at the location of the missing data point and a magnitude of zero elsewhere. If the missing observation which occurs at time t_1 is set equal to zero in the original data set (i.e. $Y_{t_1} = 0$), then at time t_1 (2) can be written as

$$-\omega_{01} = \bar{Y} + N_{t1} \qquad (3)$$

Thus, after obtaining a maximum likelihood estimate (MLE) $\hat{\omega}_0$ of ω_0, the estimate of the missing data point is given by $-\hat{\omega}_{01}$ from (3). Notice that the noise term in (3) accounts for the autocorrelation structure of the time series when the value of the missing data point is determined.

The model in (3) can be expanded to handle the situation where there are more than one missing observation. If there are I_2 missing data points in a time series the model is given as

$$y_t - \bar{y} = \sum_{j=1}^{I_2} \omega_{0j}\xi_{tj} + N_t \qquad (4)$$

where ω_{0j} is the parameter of the jth transfer function and ξ_{tj} is the jth intervention series which is assigned a value of one where the jth data point is missing and a value of zero at other times. When, in addition to I_2 missing observations there are also I_1

external interventions in a given time series, then (1) and (4) can be combined to obtain

$$y_t - \bar{y} = \sum_{i=1}^{I_1} v_i(B)\xi_{ti} + \sum_{j=I_1+1}^{I_1+I_2} \omega_{0j}\xi_{tj} + N_t \qquad (5)$$

where the first summation term on the right hand side of (5) accounts for the I_1 external interventions, the second summation term allows for the I_2 missing data points and the noise term N_t reflects the seasonal autocorrelation structure which is present in the data.

In many regions of the world climatic data such as precipitation and temperature time series are available in addition to the river flow data. For a situation where there are C_1 covariate or input series and no missing data points or interventions, a transfer function-noise model may be written as

$$y_t - \bar{y} = \sum_{k=1}^{C_1} v_k(B)(x_{tk} - \bar{x}_k) + N_t \qquad (6)$$

where $v_k(B)$ is the kth transfer function, $x_{tk} = G(X_{tk})$ is an appropriate Box-Cox transformation of the kth covariate series and $\bar{x}_k$ is the mean of the x_{tk} series.

River flow data in the Canadian Arctic is especially amenable to analysis using the methods discussed in this section. Sometimes severe ice conditions preclude the measurement of flows at certain times in the winter. In addition, the extensive development of the Canadian north may alter the hydrological characteristics of the region. Finally, temperature and precipitation records are usually available for longer periods of time than the river flow time series. Consequently, by combining (5) and (6) a viable, comprehensive and flexible model for analyzing hydrological time series which often arise in practical situations is given by

$$y_t - \bar{y} = \sum_{i=1}^{I_1} v_i(B)\xi_{ti} + \sum_{j=I_1+1}^{I_1+I_2} \omega_{0j}\xi_{tj} + \sum_{k=I_1+I_2+1}^{I_1+I_2+C_1} v_k(B)(x_{tk} - \bar{x}_k) + N_t \qquad (7)$$

Consider the case where y_t is the time series of transformed flows, there are two covariate series comprised of temperature and precipitation, various data points are missing in the y_t series and there are one or more external interventions due to man-induced causes such as construction of a reservoir and urban development. This situation can be readily modelled using (7). In addition, if the temperature and precipitation records are longer than the river flow time series, (7) can be used for back-forecasting to determine the flow records for the periods of time for which the covariate records do not overlap with the river flow time series.

MODEL CONSTRUCTION

No matter what type of model is to be fit to a given set of time series, it is recommended to adhere to the identification, estimation and diagnostic check stages of model development [Box and Jenkins, 1970; Hipel and McLeod, 1979]. The purpose of the identification stage is to ascertain the parameters which should be included in the model. For the case of the general intervention model, the model in (7) can be qualitatively written as

$$\text{output} = \text{interventions} + \text{missing data terms} + \text{inputs} + \text{noise} \qquad (8)$$

The form of the transfer function for the missing data points is given in (4), (5) and (7) while Hipel et al. [1977b] describe methods for determining the parameters required in the transfer functions for the intervention components and the input series, and also the number of autoregressive (AR) and moving average (MA) parameters which are needed in the noise component. Many of the identification methods can be most readily interpreted by using graphical displays. When these simple identification tools are used in conjunction with a sound physical understanding of the problem, usually the form of the complete intervention model can be quickly identified. In addition, if there are single input series such as precipitation and an output series such as river flows, the methods of Haugh and Box [1977] and Box and Jenkins [1970, Ch. 11] may be used to determine the type of transfer function which should be utilized.

The parameters in the intervention model in (7) are estimated using the method of maximum likelihood [McLeod, 1977]. If a transformation of the output variable is required, an estimate can be obtained for the Box-Cox power transformation parameter. In addition, the value of the Akaike Information Criterion (AIC) [Akaike, 1974] can be calculated for use in model discrimination [Hipel et al., 1977a; McLeod et al., 1977].

For the intervention model it is assumed that the residuals of the noise component in (1) to (8) are independent, homoscedastic and perhaps normally distributed. At the diagnostic check stage various tests can be carried out to determine the adequacy of the fitted model. The residual autocorrelation function can be plotted along with the 95% confidence limits [McLeod, 1978] to ascertain whether or not the residuals are white. If the residuals are correlated then appropriate model modifications can be made by repeating the three stages of model construction. If diagnostic testing reveals that the residuals are heteroscedastic and do not follow a normal distribution, these problems can often by rectified by transforming the output series and perhaps any input data sets by a suitable Box-Cox transformation [Hipel et al., 1977a; McLeod et al., 1977].

THE FOREST FIRE MODEL

The Pipers Hole River is located in the southeastern part of the province of Newfoundland, Canada, and covers an area of 829 km^2. The drainage area consists of 88 km^2 of lakes, 176 km^2 of bog, 461 km^2 of barrens and 104 km^2 of forest. There is no human habitation in the basin and no access road to the interior.

The Pipers Hole River drains into the head of Placentia Bay which forms part of the Atlantic Ocean along the coast of Newfoundland. A gauging station located near the mouth of the river has been in continuous operation since 1953 and records the natural runoff from 777 km^2 of the drainage area. Stream flow time series in Newfoundland are available from Environment Canada [1974].

A major fire destroyed an expanse that included 85% of the Pipers Hole drainage basin during the period of August to October, 1961. Besides fir and various deciduous species, spruce was the major tree type in the basin prior to the fire. The fire incinerated most of the forest and all other forms of vegetation within its path. The shallow soil mantle in the lower reaches of the basin was destroyed and consequently exposed surface boulder over most of the area.

A unique application of intervention analysis was presented by Hipel et al. [1977c] when they developed an intervention model which incorporated the effects of the forest fire upon the average monthly flows of the Pipers Hole River. A forest fire can have transitional effects upon river flows that must be included in an intervention model. Since the surroundings are denuded of all vegetation, initial sudden changes occur in the flow regime of the river. However, over the years as the vegetation recovers, the river flows gradually revert to their previous state.

The Bay du Nord River which is located 69 km west of the Pipers Hole River was untouched by the 1961 fire. Flows records have been tabulated on a continuous basis since 1952 and at the location of the measuring gauge the Bay du Nord River drains an area of 1176 km^2. The vegetation cover of the Bay du Nord basin is similar to that of the Pipers Hole River basin prior to the fire and due to their geographic proximity the climate of these two basins is the same. Therefore, the Bay du Nord flows are suitable for comparison to those of the

Pipers Hole River. By including the Bay du Nord flows as a covariate series in the intervention model for the Pipers Hole River, flow changes that are not due to the forest fire but are a result of climatic conditions will be automatically accounted for. In this way the intervention component of the intervention model will only describe changes which resulted from the fire.

Large river flows in Newfoundland occur in the spring when the snow melts. Consequently, in terms of average monthly flows, a forest fire may cause significant alterations in flow patterns during the spring months. Examination of identification plots suggested by Hipel et al. [1977b] revealed that the flows for March and April may be changed by the fire. The flows in these months appeared to increase immediately after the fire followed over the years by a steady decrease towards former levels. Because this type of variation did not occur in the Bay du Nord river, this suggested that the changes in the Pipers Hole flows, excluding intrinsic random variation, were due solely to the forest fire [Hipel et al., 1977c].

Based upon the foregoing information and after following the three stages of model construction, the intervention model for the forest fire was designed as [Hipel et al., 1977c]

$$y_t - \bar{y} = \frac{\omega_{01}}{(1-\delta_{11}B^{12})}\xi_t + \omega_{02}(x_t-\bar{x}) + N_t \qquad (9)$$

where y_t is the natural logorithm of the Pipers Hole River flows from January 1953 to December 1973, $\bar{y}$ is the mean of the y_t series, ξ_t is the intervention series which has a value of unity for March and April 1961 and a magnitude of zero elsewhere, ω_{01} and δ_{11} are the parameters of the transfer function for the intervention series, x_t is the series of natural logarithms of the Bay du Nord River flows, $\bar{x}$ is the mean of the x_t series, ω_{02} is the parameter for the transfer function of the Bay du Nord flows, and N_t is the seasonal Box-Jenkins noise term which has one nonseasonal MA parameter in addition to one seasonal MA parameter. The first term on the right hand side of (9) is the intervention component of the forest fire model and it accounts for the effects of the forest fire. Because the MLE of δ_{11} is 0.946 and hence is less than unity, when $\omega_{01}/(1-\delta_{11}B^{12})$ is expanded as an infinite series it can be seen that the transfer function models the gradual return of the spring flows to previous levels due to vegetation regeneration. An added advantage of having the covariate series in the intervention model is that it removes the need for differencing operators in the seasonal noise term. In situations where a covariate data set is not available and differencing operators are required in N_t, deseasonalization of the given time series may remove the necessity for differencing [Hipel et al., 1977b]. As explained by Hipel et al. [1977b] differencing should be avoided if the model is to be used for simulation and when it is necessary to calculate expected values in (9).

By taking antilogarithms and expected values of (9) before and after the intervention, the percentage increase in mean level of the runoff for March and April after the fire is

$$\% \text{ increase} = (\exp (\omega_{01}\delta_{11})^{(\text{date} - 1962)} - 1)100 \qquad (10)$$

where the MLE of ω_{01} and δ_{11} are 0.392 and 0.946 respectively and date indicates any year after 1961. The percentage increases in flows for March and April after the fire are calculated using (10) and listed in Table 1. As the vegetation recovers, it is expected that the increase in average flows will subside over the years and by the year 2000 it should only be about 4.5% greater than it was before the fire. This argument is, of course, only valid if the Pipers Hole River basin is not altered by any other major natural or man-induced interventions in the interim. New interventions can be readily incorporated into a revised intervention model.

TABLE 1

Percentage Increase in the Spring Runoff After the Fire.

DATE	% INCREASE IN SPRING RUNOFF
1962	47.95%
1963	44.83
1964	41.93
1965	39.25
1966	36.76
1967	34.44
1968	32.29
1969	30.29
1970	28.42
1971	26.68
1972	25.06
1973	23.55
1974	22.13
1975	20.81
1976	19.57
1977	18.41
1978	17.32
1979	16.31
1980	15.35
1990	9.50
2000	4.50

CONCLUSIONS

As exemplified by the forest fire application, the intervention model constitutes a flexible approach to stochastic modelling. In fact, intervention analysis may prove to be the most useful and comprehensive mathematical tool which has been introduced into hydrology in the recent past. Some of the important applications of intervention analysis include:

(1) modelling the effects of man-induced and natural interventions upon the mean level of a time series,

(2) describing dynamic relationships between output and input time series,

(3) estimating missing observations,

(4) using back-forecasting to determine missing records during the first time period of the output series for which the input series is known,

(5) designing data collection schemes,

(6) forecasting,

(7) simulating possible realizations of the output [McLeod and Hipel, 1978].

The foregoing applications and also an account of the mathematical structure and method of application of the intervention model, are presented in detail in the forthcoming book of Hipel and McLeod [1979].

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HYDROLOGIC EXTREME VALUE MODEL AND RISK ANALYSIS APPLIED TO FLOOD PLAIN MANAGEMENT

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ABSTRACT

The increase of human activities in flood regions tend to press competent authorities to realize some planning programs, consisting of building hydraulic structures in flood plains. The results were not the ones anticipated because the number of catastrophic floods (floods causing cracks and damages to hydraulic structures) increased with the building of hydraulic structures to avoid floods.

With this fact, there is a need to consider a solution where the rational planning of flood plains will be done taking into consideration two points:

i) zoning: which is a means to control the development of flood plains by separating them in different zones with a given risk;

ii) insurance: after the completion of the zoning phase, a private or governmental agency should establish some insurance premium against flood damages by taking into consideration the probable risk of each zone.

The creation of these zones are similar to the realization of a relationship between the return period or the occurrence of a given flood and the flood level in meters of water. A mathematical model based on the extreme values theory in stochastic processes has been established where the distribution function of these maximum exceedances in an interval of time $(0,t]$ could be found.

In the present paper bayesian and maximum likelihood approaches are followed in order to find the distribution function of the exceedance for a given return period when uncertainty arises in the parameters of the distribution function describing the model. The risks encountered in evaluating this exceedance using different lengths of period of record are compared.

1. INTRODUCTION

Essential to the management of flood plains and water resources are three broad considerations: technical, social, and economic.

In the present study we are interested in problems relating to the management of flood plains considering them as an integrated system formed of hydrologic, hydrodynamic, physical, and economic components (figure 1).

The approach taken is that of digital simulation which regards flood phenomena and their impact on flood plains as a complex system that permits the estimation of hydrologic characteristics (flood stage, return period,...) for every economic unit in various activity sectors (residential, agricultural, industrial, commercial,...). This approach makes possible the establishment of flood damage relations which can be applied at a particular time for a particular locality.

2. THEORETICAL CONSIDERATIONS

There are three common approaches to the problem of flood estimation. The oldest approach is based on empirical formulas that relate flood discharge to physical factors, the most important of which is the watershed area. The second approach is completely statistical. It consists of fitting flood discharge data to theoretical probability density functions. In the third approach; the most recent among the three approaches, and the one taken in the present study, results in the theory of stochastic processes play an essential role.

Given the discontinuity in time of flood phenomena, the exceedances, or discharges above a certain base level can be obtained by applying the following model on the river hydrograph

$$\zeta_\nu = \begin{cases} 0 & Q_\nu \leq Q_b \\ Q_\nu - Q_b & Q_\nu > Q_b \end{cases}$$

where Q_b is the base flow rate (maximum discharge within the river channel).

Q_ν is the flood peak discharge at time $\tau(\nu)$.

ζ_ν is the exceedance at time $\tau(\nu)$ (non negative).

On considering only events above the base discharge level, a stochastic process, discrete in time, is defined on the interval $(0,t]$, (figure 2). One is able to separate two distinctive properties in such a bivariate process:

Dynamic properties:

i) A set $\{\tau(\nu);\ \nu = 0,1,2,\ldots\} \subset (0,t]$ representing the time of occurence of exceedances ζ_ν

ii) The number of exceedances in $(0,t]$:

$$\eta(t) = \mathrm{Sup}\{\nu;\ \tau(\nu) \leq t\}$$

Quantitative properties:

i) The values of exceedances ζ_ν

ii) The largest exceedance in $(0,t]$:

$$\chi(t) = \mathrm{Sup}\{\zeta_\nu;\ \tau(\nu) \leq t\}$$

2.1 *Analysis of the dynamic properties*

Let E_ν^t denote the event that there are exactly ν exceedances in the interval $(0,t]$:

$$E_\nu^t = \{\eta(t) = \nu\} = \{\tau(\nu) \leqslant t < \tau(\nu+1)\} .$$

Under certain assumptions, the set $\{\tau(\nu);\ 0 < \tau(\nu) \leqslant t\}$ can be shown to define a non-homogeneous poisson process on $(0,t]$. The associated time dependent parameter is:

$$\Lambda(t) = E\,[\eta(t)] .$$

This is a non-decreasing function in time, representing the mean number of exceedances in $(0,t]$. It is governed mainely by seasonal and annual changes in weather conditions.

The event E_ν^t follows the poisson density function:

$$P(E_\nu^t) = \frac{[\Lambda(t)]^\nu}{\nu!}\,\mathrm{Exp}\,[-\Lambda(t)] . \qquad (1)$$

2.2 *Analysis of quantitative properties*

Let the distribution function of the largest exceedance in $(0,t]$ be denoted by:

$$F_t(x) = p\{X(t) \leqslant x\} .$$

By considering exceedances in an interval of time $[T_{k-1},T_k]$ to be independent and identically distributed with distribution function $H_k(x)$, $F_t(x)$ follows:

$$F_t(x) = \exp\{-\sum_{n=1}^{k-1} [\Lambda(T_n) - \Lambda(T_{n-1})][1-H_n(x)] - [\Lambda(t) - \Lambda(T_{k-1})][1-H_k(x)]\} \qquad (2)$$

for $k = 1,2,3,\dots$ and $t \in (T_{k-1}, T_k]$.

Rousselle (1972) found that the exponential distribution function fits adequately flood exceedances on many rivers:

$$H(x) = p(\zeta_\nu \leq x) = 1 - e^{-\beta x} \quad , \quad \beta > 0 . \tag{3}$$

Two cases are considered:

i- The exceedances ξ_ν are independent and identically distributed during an interval of time of one year (Zelenhasic, 1970) (Model I):

$$F_t(x) = \exp\{-\Lambda(t)\, e^{-\beta x}\} . \tag{4}$$

ii- The exceedances are independent and identically distributed during an interval of time of one season (Rousselle, 1972) (Model II):

$$F_{t_4}(x) = \exp\{-\Lambda(T_1)\, e^{-\beta_1 x} - [\Lambda(T_2) - \Lambda(T_1)]\, e^{-\beta_2 x} - [\Lambda(T_3) - \Lambda(T_2)]\, e^{-\beta_3 x} - [\Lambda(T_4) - \Lambda(T_3)]\, e^{-\beta_4 x}\} \tag{5}$$

with $\Lambda(T_1)$, $[\Lambda(T_2) - \Lambda(T_1)]$, $[\Lambda(T_3) - \Lambda(T_2)]$, $[\Lambda(T_4) - \Lambda(T_3)]$ representing the mean number of exceedances for each season and β_1, β_2, β_3, β_4 are the respective parameters of the exponential distribution function associated with each season.

2.3 *Determination of the return period*

In the design of a hydraulic structure, one has to select its capacity, which must be accommodated to the existing conditions. This selection is based on the return period T of a certain level x, i.e. the mean number of years required to obtain an exceedance greater than x.

Let $N_x = \inf\{\nu; X_\nu > x\}$ where X_ν is the largest exceedance in the year ν. It follows that:

$$p(N_x = n) = [F_{t^*}(x)]^{n-1}[1 - F_{t^*}(x)] \tag{6}$$

t^* denoting the one year period, and

$$\begin{aligned} E(N_x) = T &= \sum_{n=1}^{\infty} n\,[F_{t^*}(x)]^{n-1}[1 - F_{t^*}(x)] \\ &= \frac{1}{1 - F_{t^*}(x)} . \end{aligned} \tag{7}$$

Study of the "design exceedance".

Let X_T be the value of x in equation (7) corresponding to a particular value of T; i.e. X_T is the "design exceedance" corresponding to the return period T.

On combining equations (4) and (7) for Model I we obtain:

$$X_T = \begin{cases} \dfrac{C + \ln\lambda}{\beta} & \lambda > e^{-C} \\ 0 & \text{otherwise} \end{cases} \tag{8}$$

where $C = -\ln\left(\ln\frac{T}{T-1}\right)$ and $\lambda = \Lambda(t^*)$.

In practice, the parameters λ and β are estimated by random variables $\hat{\lambda}$ and $\hat{\beta}$ respectively, which implies that X_T is a random variable. To find the probability density function (pdf) of X_T one has to proceed by finding the pdf's of $\hat{\lambda}$ and $\hat{\beta}$. This is done by two methods:

i) Bayesian techniques yield the two parameter Gamma as a pdf of both $\hat{\lambda}$ and $\hat{\beta}$ (Rousselle and Hindié, 1975).

ii) Maximum likelihood estimates $\hat{\lambda}$ and $\hat{\beta}$ can be shown to have the following density function respectively:

$$f_n(k) = p(\hat{\lambda} = k) = \frac{e^{-\Lambda(T_n)}\,[\Lambda(T_n)]^{nk}}{(nk)!} \qquad k = 0, \frac{1}{n}, \frac{2}{n}, \ldots$$

$$g_n(b) = \frac{e^{-\Lambda(T_n)}}{b} \sum_{m=1}^{\infty} \frac{[m\beta\Lambda(T_n)\; e^{-\beta/b}/b]^m}{m!\;(m-1)!} \qquad b > 0 \tag{9}$$

the subscript n denoting the number of years of record. Hence the density functions of X_T:

$$\Psi_n(x) = \int_{e^{-C}}^{\infty} f_{1,n}(\lambda)\cdot f_{2,n}\left(\frac{C + \ell n\ \lambda}{x}\right) \cdot \frac{C + \ell n\ \lambda}{x^2}\, d\lambda \quad \text{(Bayesian)} \tag{10}$$

where $f_{1,n}$ and $f_{2,n}$ are the Gamma density functions of $\hat{\lambda}$ and $\hat{\beta}$ respectively.

$$\Psi_n(x) = \begin{cases} \displaystyle\sum_{k>e^{-C}} \frac{(\beta nk)^{nk}\, x^{nk-1} \exp\left[-\beta nkx/(C + \ell n\ k)\right]}{(C + \ell n\ k)^{nk}\, \Gamma(nk)} \cdot f_n(k) & x > 0 \\[2ex] 1 - \displaystyle\sum_{k>e^{-C}} f_n(k) & x = 0 \quad \text{(Maximum likelihood)} \end{cases} \tag{11}$$

On combining equations (7) and (5) (Model II), we get:

$$T = \frac{1}{1 - \exp\left(-\sum_{i=1}^{4} \lambda_i e^{-\beta_i x}\right)} \tag{12}$$

where λ_i, i = 1,2,3,4 is the mean number of exceedances in the i^{th} season.

The pdf's of $\hat{\lambda}_i$ and $\hat{\beta}_i$, $i = 1,2,3,4$ are similar to those given in (9). Hence the density function of X_T:

$$\Psi_n(x) = \int_0^\infty \int_0^\infty \int_0^\infty \theta_n(x,b_2,b_3,b_4)\ db_2\ db_3\ db_4 \tag{13}$$

θ_n being the joint density function of X_T, $\hat{\beta}_2$, $\hat{\beta}_3$ and $\hat{\beta}_4$:

$$\theta_n(x,b_2,b_3,b_4) = \sum_{k_1} \sum_{k_2} \sum_{k_3} \sum_{k_4} \varphi_n(x,b_2,b_3,b_4 | \hat{\lambda}_i = k_i,\ i = 1,2,3,4) \prod_{i=1}^{4} f_{i,n}(k_i)$$

All four summations are on the set $\{0, \frac{1}{n}, \frac{2}{n}, \ldots\}$ with the restriction: $\sum_{i=1}^{4} k_i > \ell n \frac{T}{T-1}$. $f_{i,n}$ is the density function of $\hat{\lambda}_i$.

2.4 *Risk determination*

Let X_0 be the design exceedance given in (8) (Model I) or by the (trial and error) solution for x in (12) (Model II). If the random variable X_T, which is the true exceedance corresponding to the design return period T, was exactly equal to X_0, then the loss encountered in the construction of a dam using X_0 as a basis would be nil. If X_0 is greater than X_T, however, the dam construction produces a loss due to over estimation. On the other hand, a loss due to under estimation is encountered when X_T is greater than X_0.

Based on the value of X_T relative to X_0, one can construct a loss function which is dependant on the nature of the hydraulic structure. A linear loss function is shown in figure 3.

If we denote by L_u and L_o the losses due to underdesign and overdesign respectively, the corresponding risk would be the sum of the expected values of these losses:

$$R_T = E(L_u) + E(L_o) .$$

3. NUMERICAL APPLICATION

The above mathematical treatment of Model I has been applied to the flood data at the St-Paul Station, Minn. on the mississipi river. Figure 5 shows the p.d.f. of the design exceedance X_T for four different return periods.

Model II has been partly applied numerically for the same station. Observed and theoretical distribution functions of the maximum flood peak exceedance flow for various combinations of seasons are shown in figure 4. An extension of this numerical application covering the risk determination is under way.

4. CONCLUSION

The social complexity of flood plain management has always impeded the establishment of a convenient flood control system. Among the aspects needed to be considered in the establishment of such a system are technical, economic, and social ones. A deep study of such aspects may lead to a regional planification system valid on the long run and acquiring the necessary flexibility to cope with the evolution of society.

The present study is a step taken in the direction of a flood control system. It is in the field of the hydrologic component of the complex flood phenominon. A stochastic approach is taken for the development of a distribution function of the largest flood exceedance in a given period of time. The following advantages are offered by this approach:

i) Models I and II presented combine the distribution functions of both flood exceedances and their time of occurrence.

ii) The random nature of floods is preserved given the broad range of probability distribution functions one is able to choose from to fit the data in a particular locality. The exponential density function, for example, has been successful choice for the "Rivière des Prairies" in Quebec (Rousselle and El-Jabi, 1976).

iii) Seasonal considerations can adjust well to the stochastic approach. A hydrologic study of snow melt floods which are common in canadian rivers is supported by such considerations.

Finally the stochastic method was proved to have a solid mathematical and physical basis (Todorovic, 1970). In addition it may have applications in the study of flood volume, in flood warning, and in other aspects of flood plain management.

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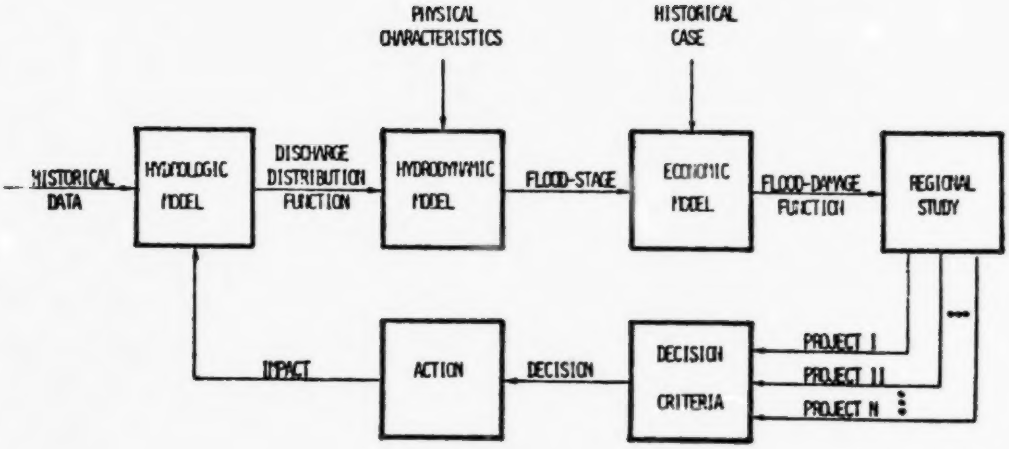
PHYSICAL CHARACTERISTICS
HISTORICAL CASE
HISTORICAL DATA
HYDROLOGIC MODEL
DISCHARGE DISTRIBUTION FUNCTION
HYDRODYNAMIC MODEL
FLOOD-STAGE
ECONOMIC MODEL
FLOOD-DAMAGE FUNCTION
REGIONAL STUDY
PROJECT I
PROJECT II
PROJECT N
DECISION CRITERIA
DECISION
ACTION
IMPACT

Fig. 1 - Schematic Description of the Flood Plain Management System.

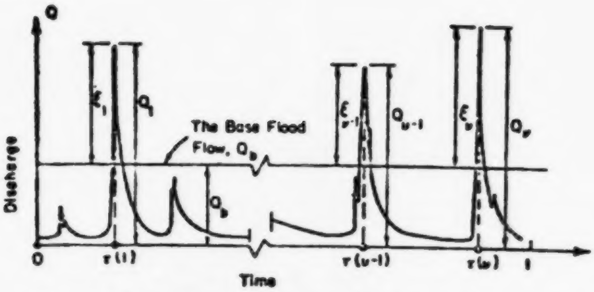

Fig. 2 - General streamflow hydrograph of instantaneous discharges at a given location of a river.

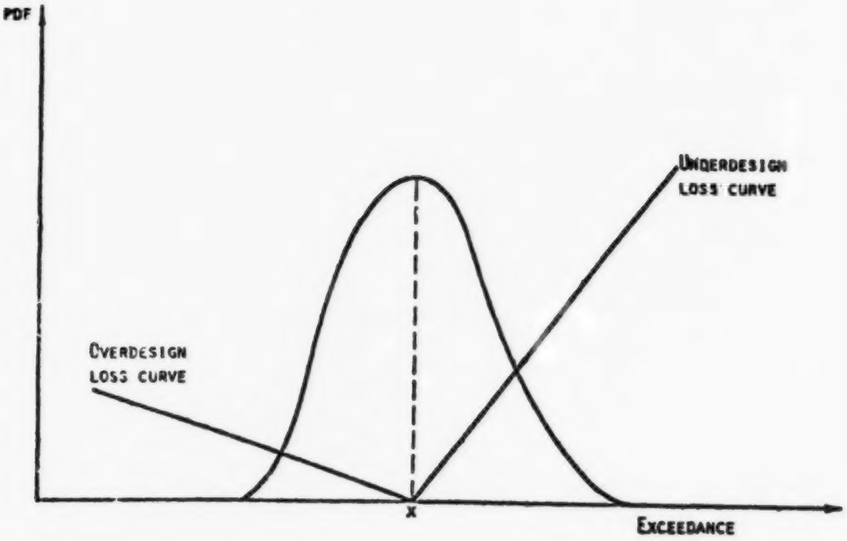

Fig. 3 - The error PDF and loss curves for errors X in flood estimates

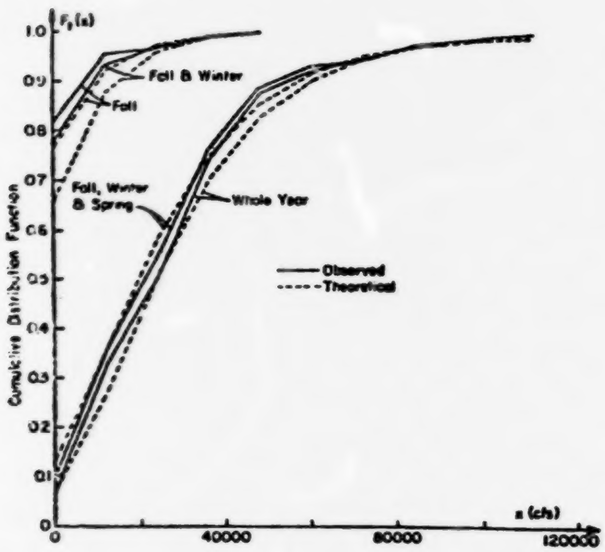

Fig. 4 - Observed and theoretical distribution functions of the maximum flood peak exceedance flow for various combinations of seasons for the Mississippi river at St. Paul, Minn.

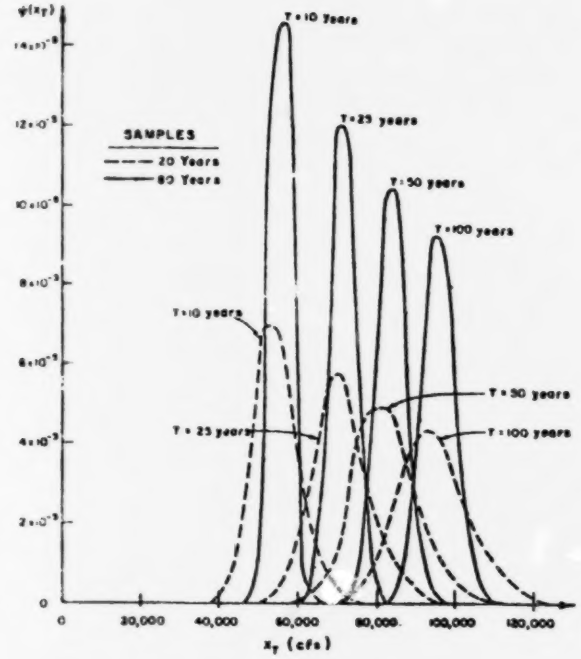

Fig. 5 - Error distribution functions of the maximum flood peak exceedance flow for various return periods samples length for the Mississippi river at St.Paul, Minn.

HYDRO CONFIGURATION MODELLING SYSTEM

D.W. Farley [1]
April 1979

RESUME

A generalized modelling package referred to as HCMS (Hydro Configuration Modelling system) has been developed for studying multi-reservoir river systems. The package is particularly applicable to basins in which there is an existing hydroelectric generating capacity and/or a potential for further development. The package is used to evaluate proposals relating to: increased reservoir storage devoted to flood protection; new reservoirs; new installed turbine capacity; environmental requirements in terms of constrained reservoir levels or minimum flows; diversions of flow; and increased channel discharge capacity. By comparing the results of different simulated operation policies and configurations with each other and with historical performance the planning team can readily assess the impact on flood protection, energy production and environmental concerns. In order to expedite the modelling phase of studies the package is endowed with a realistic optimal seeking solution routine. This optimal seeking solution routine employs a linear programming sub-routine available from the IBM package SL-MATH. As opposed to other applications of linear programming to water resource systems studies, the optimal seeking solution routine of this model does not assume perfect hindsight of time series data, nor is it restricted to critical periods of record to conform with dimensionality requirements common to other optimization models.

Problems in Applying Optimization Methods to Water Resources Studies

In water resources systems studies the planning team works with time series data which hopefully comprise a lengthy period of historical records. If such records are short, then the planning team must lengthen the records with synthetic data obtained by correlation or stochastic modelling techniques. Systems studies are based on the premise that the statistical properties of the hydrological data sets used in the evaluations are representative of future hydrological records which will be experienced during the life of the proposed project. Engineering projects usually have design lives exceeding 30 years, thus it is important that a planning horizon of at least 30 years be considered in the studies.

Linear programming and dynamic programming are probably the two most popular optimization techniques applied to water resources studies. Dynamic programming can be applied effectively to long periods of record, however it cannot cope with more than one or two decision variables within each sequential time period in the planning horizon. Hence, multi-reservoir studies preclude the practical application of dynamic programming. Linear programming, as its name implies, requires linear relationships which would seem to be inconsistent with reservoir river systems defined by such non-linearities as: stage-storage curves; stage-discharge curves; and power-discharge curves. Another problem in the application of linear programming in past systems studies (Draper, 1976) is that the dimensions of the decision variables exceed the capabilities of the existing linear programming packages, even though some of these packages are so powerful that they can handle more than 8,000 equations simultaneously. These packages are fine for business applications, but in water resources the planner quickly encounters difficulty with a multi-reservoir system and a long planning horizon. For instance, a 15 reservoir system with a planning horizon of 30 years and a unit time period of analysis of one week would require a minimum of 23 400 equations. The standard procedure in North America in accommodating the dimensionality problem has consisted of the following three manoeuvres: (1) increase the duration of the unit time periods to at least one month; (2) select a critical period of record of less than two years for the planning horizon of the linear programming solution; (3) test the results by trial and error in a specially designed simulation model that can accommodate the entire planning horizon. The optimization model in these past studies is often referred to as a screening model. There are several problems that are immediately apparent in carrying out the above three manoeuvres. The conventional use of a critical period is inadequate because no probability or frequency can be attached to the occurrence of the critical period. Lengthening the unit time period to one month renders the analysis almost useless for flood frequency analysis and environmental assessment. Instead of an optimal solution being computed by the linear programming technique, the solution is really being found by trial and error use of the simulation model which is a procedure costly in time and computer usage. Furthermore, this approach assumes perfect hindsight over the critical period which does not represent reality when one takes into account present forecasting capabilities. Lastly, non-linearities during the critical period analysis are ignored.

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At the beginning of Ottawa River Regulation Studies in 1975 (Montreal Flow Regulation Committee, August 1976) a goal was set to design an optimal seeking solution simulation model which could be used for the entire planning horizon and yet effectively overcome the above problems. It is felt that this goal was met when under the auspices of the Montreal Flow Regulation Committee the Hydro Configuration Modelling System (HCMS) was developed. The basic concepts of the HCMS package were derived from a model developed in Greece (Valvis, P.Z., August 1974). However, the HCMS package differs from the Greek model with respect to the objective function formulation and in the method of application of a linear programming sub-routine.

HCMS Model Overview

In addition to overcoming the problems mentioned above, the major objective of the development of the HCMS model was to provide an easy-to-use package for:

(1) generating models of hydrologic/hydroelectric configurations

(2) manipulating these models so as to assess the impact on energy production, discharges and levels: increased reservoir storage devoted to flood protection; new reservoirs; new installed turbine capacity; environmental demand relating to constrained reservoir levels; diversions of flow; and increased channel discharge capacity.

(3) comparing the results of different simulated operation policies with each other and with historical performance.

The optimal seeking solution routine of the model is guided by the following objectives:

(a) Operate the reservoir-river system in such a way as to satisfy constraints on reservoir elevations relating to flood control, navigation, flow augmentation, and recreational requirements.

(b) Maximize the primary power of the system. That is to say, produce the maximum continuous power generation that can be guaranteed for the period of record under study. An option which is often used in the model is to satisfy the constraint of a stated primary power value for the system.

(c) Maximize the potential energy for each unit time period. This has the effect of keeping water upstream for future use in power generation and low flow augmentation.

The potential energy is described mathematically as follows:

$$\Sigma P_i \; HR_i \quad \text{(units- MW.SEC)} \tag{1}$$

where: HR_i = reservoir level elevation of reservoir i

$$P_i = \Sigma CCPj/DHRDV_i \tag{2}$$

- j represents the power houses below i
- CCP is the characteristic coefficient of the power house in megawatts/cfs
- and $DHRDV_i$ is the rate of change of head with reservoir volume at reservoir $_i$

The model will do everything possible to satisfy the reservoir level constraints specified under objective (a) above. However, it will allow a level constraint to be violated if an infeasibility would otherwise occur. An example of an infeasibility would be a specified reservoir level constraint being physically unachievable given the maximum discharge capacity of the outlet and the relatively high inflow. Another example would be the inflow to a reservoir not being sufficient to satisfy a minimum level recreational constraint. These violations of level constraints are accomplished within the model by introducing auxiliary variables designated as AHR and AHR'. Another constraint that is allowed to be violated is the specified value for primary power. The model in this case uses another variable designated as auxiliary power (AP) to take care of this violation. The model is forced to try and keep these auxiliary values at zero by applying large negative cost coefficients which severely penalize the objective function. Violations of specified level constraints are flagged in the output in order to inform the planner how often his specified level constraints are physically unachievable. Figure 2 illustrates the Simplex Tableau in which all the above variables including discharges and spills appear. Constraints which are never allowed to be violated are absolute maximum levels for reservoirs (otherwise, overtopping would occur), and maximum flow through turbines and spillways.

The model accepts the following 3 types of structures in the configuration:

(Type 1) Simple storage i.e. a storage reservoir with a controlled outlet with no power generating units

(Type 2) Power plant with reservoir storage

(Type 3) Run of river generating plants

Figure 1 illustrates a configuration containing all three types of structures. Any number of these plant types can be linked together by river sections of varying lengths. Lag time for water to flow down these links can be specified as a constant for each link with a maximum value equal to the unit time period.

Given the network configuration, the model generator automatically generates the LP model which consists of 3 sections:

(a) continuity equations
(b) prime power achievement constraint
(c) objective function

The model consists of two major elements - a "Driver" and a "Linear Programming Package". DOSPB a sub-routine of the IBM SL-MATH package is used for the linear programming (LP) solution. The model can use any unit time period (three days to one month) providing that the unit time period exceeds the longest lag time used in the routing calculations. A ten-day unit time period was chosen for the Ottawa River study (Montreal Flow Regulation Committee, 1976).

Because of the dimensions of the problem and the non-linear nature of the relationships, the problem is modularized as follows:

1) The total period of record (say 30 years) is broken down into the selected unit time periods.

2) A guess is made as to the achievable prime power over the 30 year period.

3) For each unit time period, the system attempts to satisfy all constraints (including prime power), while maximizing a "future power potential" function. This is achieved through the LP model which is updated with relinearized coefficients and bounds, and subsequently solved. The Driver analyses the LP output, makes adjustments to reservoir elevations which were computed from a linear relationship, and computes a new set of coefficients and bounds to be used in the LP solution for the next time period.

4) An iteration control routine monitors the achieved power and adjusts the target in such a way that a good approximation to the maximum achievable prime power is achieved after a small number of iterations (i.e. passes through the entire period of record).

The output from the last (i.e., "optimal") pass is stored on a file and can be printed or plotted by utility programs as required. The output includes the following for each unit time period: reservoir elevations at the end of each period; mean discharges; mean power generation at each plant; observed bounds; violations of bounds; and mean spills at type 2 and 3 plants.

Continuity Equations

For each plant there is a steady-state equation which states mathematically that water entering a plant must be either stored by that plant or pass through the plant. The form these equations take for each plant type are expressed below:

TYPE 1

$$(HR_i - AHR_i + AHR_i') + F_i QT_i - F_i \sum_j X_j QT_j - F_i \sum_j X_j SP_j$$

$$= HRP_i + F_i Q_i + F_i \sum_j Y_j QTP_j + F_i \sum_j Y_j SPP_j$$

TYPE 2

$$(HR_i) + F_i QT_i + F_i SP_i - F_i \sum_j X_j QT_j - F_i \sum_j X_j SP_j$$

$$= HRP_i + F_i Q_i + F_i \sum_j Y_j QTP_j + F_i \sum_j Y_j SPP_j$$

TYPE 2

$$QT_i + SP_i - \sum_j X_j QT_j - \sum_j X_j SP_j$$

$$= Q_i + \sum_j Y_j QTP_j + \sum_j Y_j SPP_j$$

where:

$X_j = (DT-LAG_j)/DT_i$

$Y_j = LAG_j/DT$

$F_i = DHRDV_i \,.\, DT$

QT_i = average discharge from the reservoir (type 1 plant) or through the turbines (type 2 or 3 plant) over the time period

IN_i = average inflow from upstream plants over the time period

Q_i = mean basin supplies entering the system between the plant and the immediate upstream plant(s)

SP_i = average spill (type 2 and 3 plants) over the time period

NOTE: The net basin supplies (Q_i) for the planning horizon are stored in a special file. These values for each unit time period must be derived from the records.

Plant Characteristics

The plant characteristics are represented as follows:

(i) Reservoir Stage Storage Curves: These curves are represented by one to three polynomial regression equations depending upon the degree of curvature. Because of the nonlinearity in these curves, the driver element of the model must compute a corrected reservoir level at the end of each unit time period.

(ii) Generator Power Versus Head: The power production for a type 2 plant depends upon both total discharge and head pond elevation. The relationships for these plants have been approximated by regression equations of the form:

$$\text{Power} = k(QT)^a \ (HR)^b - K.SP \tag{3}$$

where k, a and b are constants and QT = turbine discharge

The reduction in power due to spill is assumed to be directly proportional to spill. This negative slope is represented by the constant K.

In some cases, power is a function of discharge and elevation at the given plant as well as stage at a downstream plant. For these cases, the equation has been extended to the form:

$$\text{Power} = k(QT)^a \ (HR)^b (HRD)^c \tag{4}$$

where HRD is the elevation of the downstream plant, and c is a constant

For a run of a river plant; $\text{Power} = k(QT)^a$ (5)

If there is a backwater effect from a downstream power plant,

$$\text{Power} = k(QT)^a \ (HRD)^b \tag{6}$$

The power produced at a Type 2 or 3 plant in a given operating period is approximated by:

$$\text{Power} = CCP \ . \ QT - K.SP \tag{7}$$

The updated calculation of CCP is performed in order to update the coefficients provided to the LP model before each unit time period run.

The maximum flow through the turbines, QTMAX, for a type 2 plant is represented by:

$$QTMAX = QTM_O + cHR \tag{8}$$

where QTM_O and c are constants

For a run of the river plant (type 3), the second term in equation disappears.

The mathematical details and the documentation for using the programs of HCMS are found in the reference by Emery, 1979.

HCMS Options

There are many options in using HCMS. For instance, prime power may not be an important consideration. In this case, the user could specify in the model input only one iteration for a given prime power value. Also, it might be found that prime power is governed for all schemes in a study by a particular critical period in the planning horizon. The iteration feature could then be employed for just the critical period. The value for prime power thus obtained can be used for the entire planning horizon with only one iteration specified.

The model assumes prior knowledge of basin supplies for only the unit time period under consideration. In Ottawa River studies (Montreal Flow Regulation Committee, 1976) the selected unit time period was 10 days. This period conformed roughly to downstream forecasting capability. If a user felt that current and antecedent conditions such as snow pack and soil moisture index justified changes to the reservoir bounds he could run the model on an annual basis. Continuously updated reservoir bounds are a feature of the Ottawa River Operational Modelling System presently under development (presentation by Nix, Group 1).

Limitations of HCMS

- Unit time period may vary from 3 days to 30 days.
- Reservoir level bounds as specified for each unit period of a year are constant throughout all the years of record unless the user decides to update the bounds for each year and run the model on an annual basis.
- Maximum discharge constraints as pertaining to floods may not be specified to the model. The user may only adjust the reservoir bounds on the reservoirs by trial and error in order to improve operating policy over the entire planning horizon. Note that with regard to discharges, there are physical limitation bounds set for turbines and spillways. These bounds can never be violated.
- Spill is assumed to have a completely linear effect on power production.

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FIG. 1

HCMS MODEL TEST SCHEMATIC

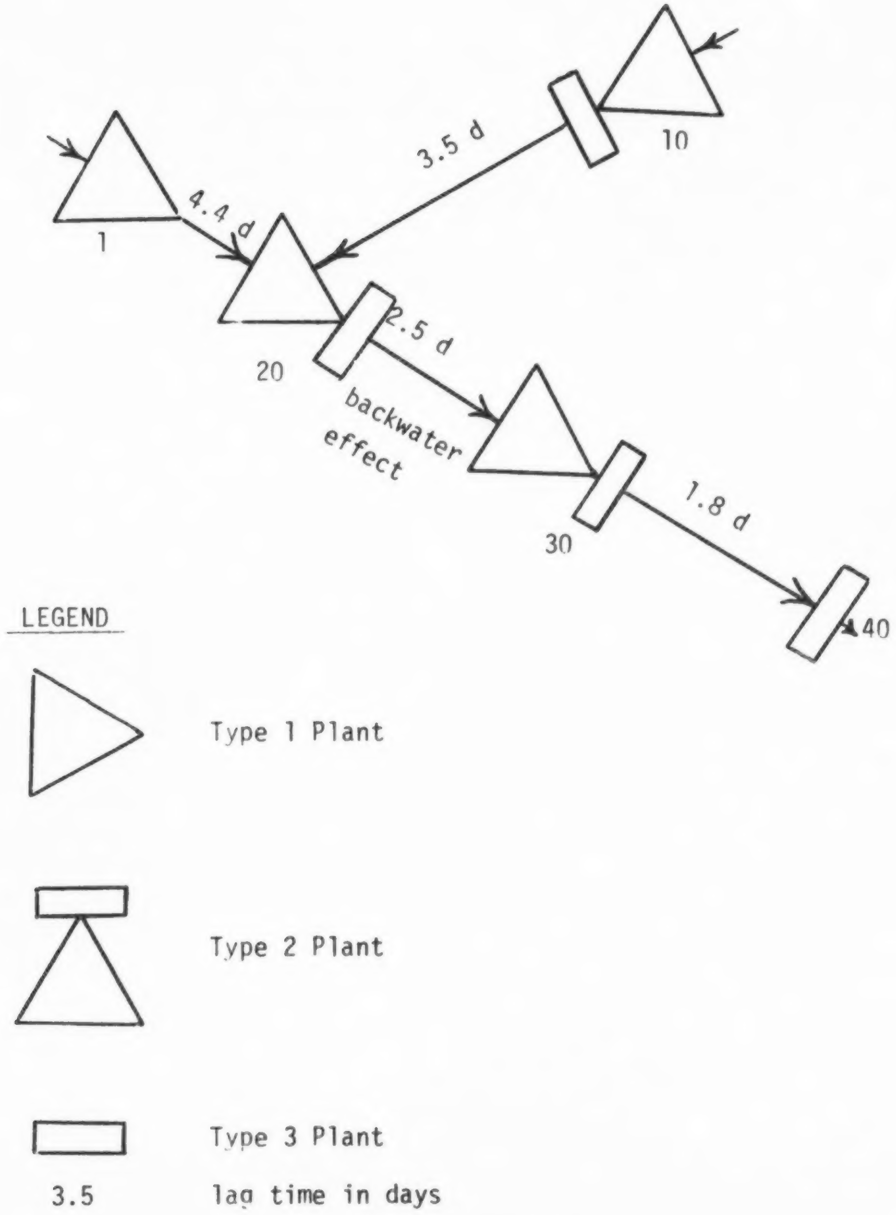
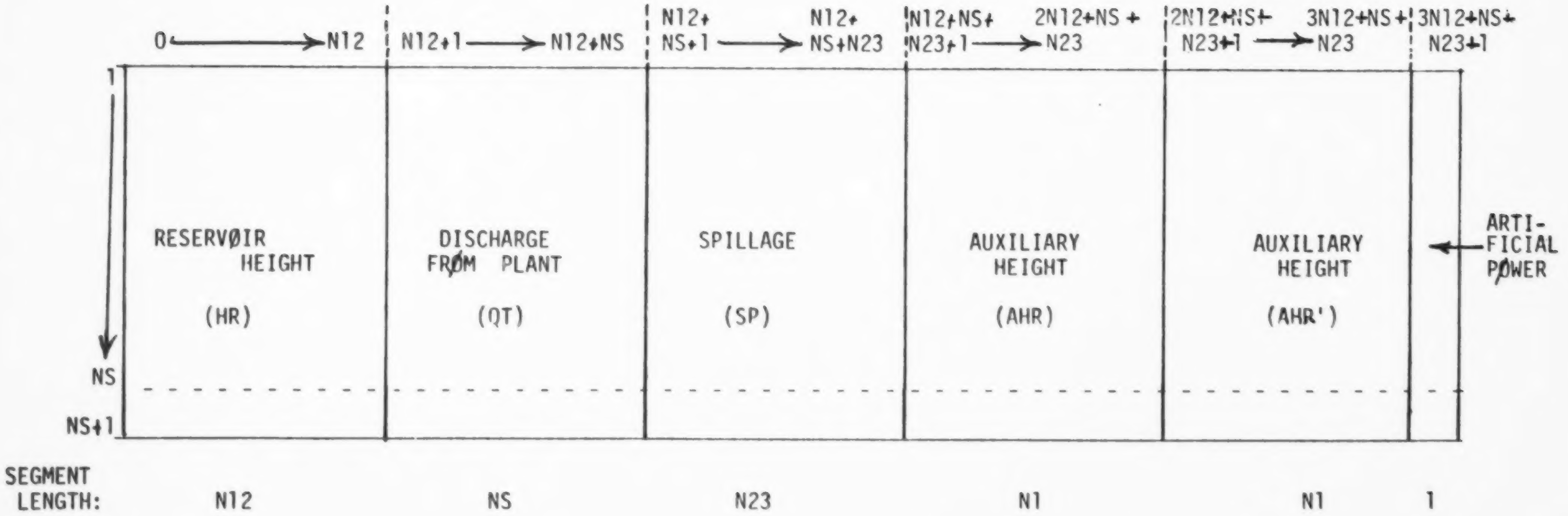

3.5 d
10
4.4 d
1
2.5 d
20
backwater effect
30
1.8 d
40
LEGEND
Type 1 Plant
Type 2 Plant
Type 3 Plant
3.5
lag time in days

FIG. 2

SIMPLEX TABLEAU FOR HCMS MODEL


0 → N12
N12+1 → N12+NS
N12+ NS+1 → N12+ NS+N23
N12+NS+ N23+1 → 2N12+NS+ N23
2N12+NS+ N23+1 → 3N12+NS+ N23
3N12+NS+ N23+1
1
NS
NS+1
RESERVØIR HEIGHT (HR)
DISCHARGE FRØM PLANT (QT)
SPILLAGE (SP)
AUXILIARY HEIGHT (AHR)
AUXILIARY HEIGHT (AHR')
ARTI-FICIAL PØWER
SEGMENT LENGTH:
N12
NS
N23
N1
N1
1

NØTE$_1$: $N12 = N1 + N2$, $N23 = N2 + N3$ where

N1 = total number of type 1 plants in the network
N2 = total number of type 2 plants in the network
N3 = total number of type 3 plants in the network

NØTE$_2$: Dimensions of TABLE A are (NR,NV)

where: $NR = NS+1 = N1+N2+N3+1$

$NV = 4N1+3N2+2NS+1$

A PROCEDURE FOR DETERMINING FLOOD FREQUENCIES FOR GREAT LAKES SHORELINE

by

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A PROCEDURE FOR DETERMINING FLOOD FREQUENCIES FOR GREAT LAKES SHORELINE

INTRODUCTION

The Great Lakes Shoreline is 17,000 kilometres in length including islands, half of which is located in Canada. As a significant portion of this shore is either already developed or being considered for further development, the need for flood level frequency analysis has become important. Flood frequency information is needed for flood-risk mapping, the design of shore structures and to estimate the benefits of flood control works.

The procedures used to calculate flood frequencies on large lakes is significantly different from the procedures used for riverine situations. Although considerable work has been done on the development of procedures for determining flood frequencies on rivers, until recently little work has been done in developing procedures for lake shorelines.

On the Great Lakes shore, flood levels depend on three factors: the mean lake level; the storm set-up; and the wave uprush. The mean lake level is a function of the water supply and hydraulic regime while both the storm set-up and wave uprush are determined by the wind regime.

Near river mouths the flood level will depend on the above mentioned lake factors and the river discharge.

This paper presents procedures which can be used to determine flood frequencies on the Great Lakes. The procedure varies according to the availability of data. The procedure is relatively simple near gauges where there are records of peak flood levels. If such data is not available, the procedure becomes more complex as the flood frequencies must be derived by calculating wind set-up and run-up distributions from climatalogic data and combining these frequency distributions with mean water level frequency distribution for the lake.

PROCEDURES FOR GREAT LAKES SHORES

Along the Great Lakes shoreline, the water level depends on a combination of factors. One important factor is the mean water level of the lake, which is a function of the water supply, the hydraulic regime, and the operation of control dams. A second factor is the wind set-up, which is dependant on wind climate, fetch length and water depth. These two factors determine the stillwater level* along the shoreline. In areas not subject to wave action, such as marshland and areas of very flat beach slope, flooding occurs only up to the stillwater level. However, in other areas wave run-up causes flooding above the stillwater level. In these locations the wave run-up must be combined with the mean level and the set-up to determine the flood level. Wave run-up is a function of the wind climate, water depth, fetch length and beach slope. All three factors, i.e. the mean lake water level, the set-up and the wave run-up, are seasonally dependent, with the highest mean lake levels occurring in early summer and highest wind set-up and wave run-up occurring in fall and early spring.

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* Stillwater level is the water level in the absence of waves.

The seasonal dependence of these three factors and the persistence of the monthly mean water level make a strict mathematical solution of their combined probability very difficult. Therefore a solution involving simulation of monthly water levels, wind set-up and wave run-up is proposed.

The method we propose for estimating the annual water level probability distribution involves the random combination of synthesized maximum monthly set-ups and run-ups with recorded adjusted monthly mean water levels to give maximum instantaneous monthly water levels. The highest value for each year is selected from the monthly values, and these annual values are plotted to give the annual water level probability distribution.

DETERMINATION OF MONTHLY MEAN WATER LEVELS

The recorded water levels needed for this analysis are the monthly mean recorded levels adjusted for existing conditions (i.e. dredging, control works). It is recommended that the levels for the period 1900 to present be used. Adjusted levels are available from the Water Planning and Management Branch, Ontario Region Office in Burlington.

DETERMINATION OF WIND SET-UP

There are two methods that may be used for determining the set-up in water level caused by winds over the lake.

The first method is the numberical calculation of set-ups based on wind speed, fetch length and water depth. The first step is the calculation of fetch length. A method for calculating fetch length is outlined in the "Shore Protection Manual, Volume 1" on page 3-29. The wind speed data required are the maximum monthly speeds for a period sufficient to cause complete set-up on the lake. For example, for sections of Lake St. Clair, this would be the maximum wind speed for a three hour period. The wind speeds need to be from the direction that will cause maximum set-up. In some locations, two or more wind directions may result in maximum set-up. In this case, the highest of the wind speeds from any of these directions is selected. At least ten years of data are necessary, and a longer period is preferable. Since the wind speeds are maximum values, they can be represented by a Gumbel distribution. For each month the wind speeds are ordered and plotted on Gumbel paper, giving the probability distribution for maximum wind speeds for each month of the year.

It is now possible to generate a period of synthetic wind speeds based on these frequency distributions. If, for example, 80 years of monthly mean water level data are available, then 80 values of wind speeds corresponding to the probability distributions can be calculated for each month of the year, resulting in one wind speed for each monthly mean water level for each year.

The set-up on the lake can be determined by the following equation given by Bruce and Clark (1966):

$$S = 0.0007 \frac{V^2F}{D}$$

where:

S = set-up, in feet,
V = wind speed, in miles per hour,
F = fetch, in miles, and
D = average water depth, in feet.

For each synthetic wind speed the corresponding set-up is determined.

COMBINING SET-UPS WITH MEAN MONTHLY LEVELS

These set-ups can now be randomly added to the monthly mean lake levels; that is, the January set-ups are randomly ordered and added to the recorded January mean water levels, and the same is done for all the other months. The result is 80 years of maximum monthly stillwater levels. From this, the maximum annual stillwater level is selected for each of the 80 years. The annual stillwater level probability distribution is obtained by plotting these values on Gumbel paper.

CALCULATION OF STILLWATER LEVELS FROM GAUGE RECORDS

A somewhat easier method of determining the stillwater level probability distribution can be used if sufficient data are available. The monthly mean lake levels are required as in the previous method, and in addition, a continuously recording water level gauge is necessary in the area for which the distribution is required. From the gauge records, the difference between the maximum instantaneous stillwater level and the monthly mean level needs to be calculated for each month for at least ten years. The resulting values are the maximum set-ups for each month. Plots of these values on Gumbel paper for each month of the year gives the monthly set-up probability distributions. As with the wind speeds in the previous method, synthetic values of set-up are determined from the distributions. These set-ups and monthly mean water levels can be combined as described in the previous method, and the steps can be followed through to give the annual stillwater level probability distribution.

CALCULATION OF WAVE RUN-UP

In some locations, the wave run-up on the shoreland is an important factor in flooding and must be taken into consideration in determining the water level probability distribution. This can be done in combination with the first method given for determining set-up. Using the synthetic wind speeds generated, the resultant wave heights and wave periods can be determined by the method given in the "Shore Protection Manual, Volume 1" on pages 3-35 to 3-51. The wave run-up can be calculated for each case by using the equation given in "Regulation of Great Lakes Water Levels, Appendix C, Shore Property":

$$Ru = 2.3\ H^{0.5} T \tan \alpha$$

where:

Ru = wave run-up, in feet

H = wave height, in feet

T = wave period, in seconds

and $\tan \alpha$ = beach slope

The set-up and run-up resulting from a given wind speed are summed to give the total rise in water level, and these rises are randomly assigned to the monthly mean water levels, giving maximum monthly run-up water levels. The highest annual value is selected for each year, and these values are plotted to give the annual peak water level probability distribution (Figure 1). This distribution can be used in the establishment of setback requirements and in the evaluation of flood damages.

PEAK ANNUAL UPRUSH WATER LEVEL
CHENAL ECARTE AT LAKE ST. CLAIR

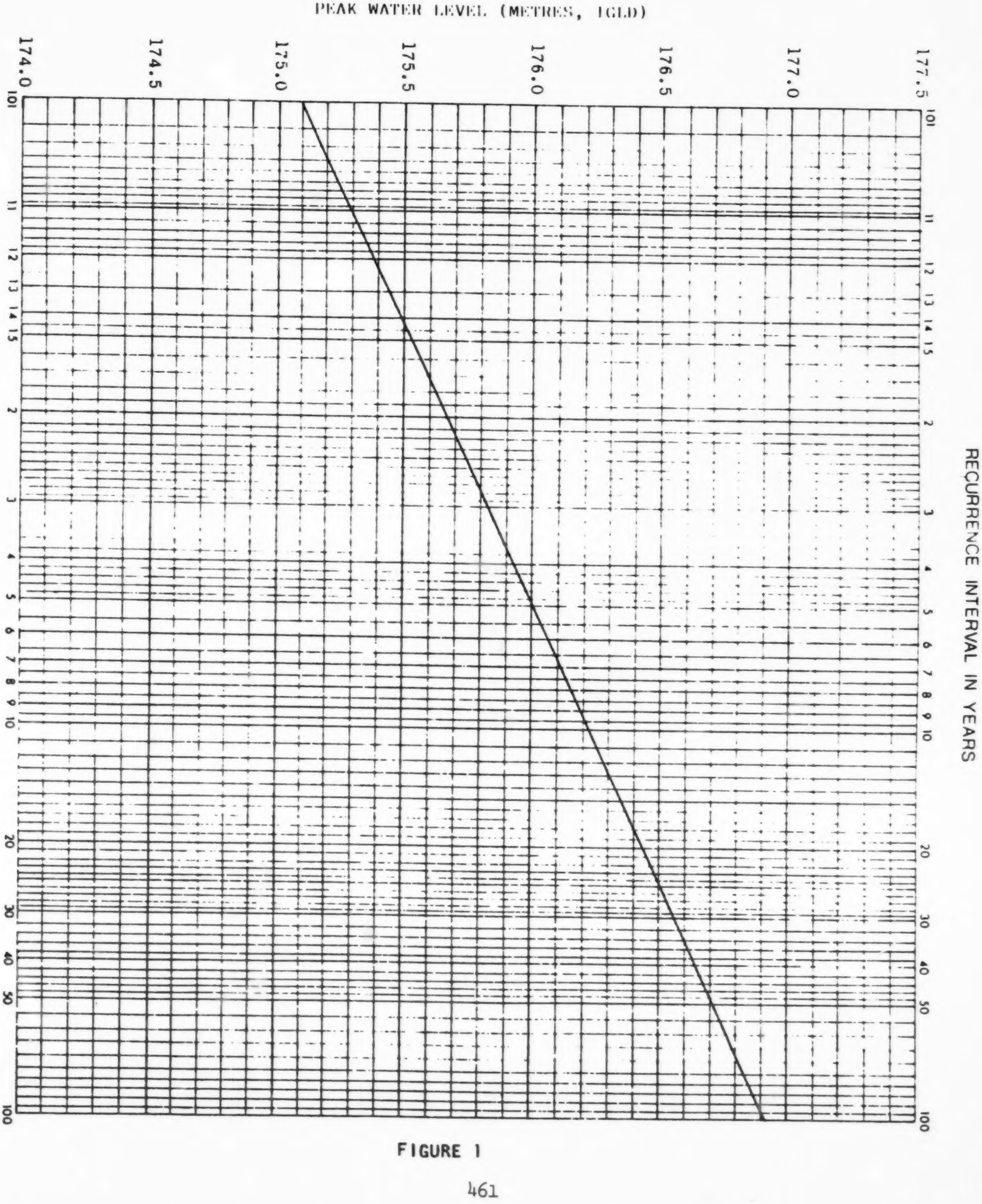

FIGURE 1

A description of the above procedure and its application to calculate water levels for a portion of Lake St. Clair is given in "Water Levels in Chenal Ecarte" (1979).

PROCEDURES FOR CHANNELS AND RIVER MOUTHS

The technique developed in the last section can be extended to incorporate the interaction of streamflows and lake levels in the vicinity of river outlets.

Traditionally, when flood plain mapping is undertaken, component streamflows and lake levels are not considered simultaneously. The combined effects of lake levels and streamflows are generally not explored. Figure 2 presents flood levels near a river mouth.

Three flood levels are shown in Figure 2: the 1:100 year lake level; the 1:100 year river flood level; and the 1:100 year level based on the combined probability of lake levels and river flows. When lake flood levels or river flood levels are computed independent of each other, the other component is not taken into consideration in the calculations. If streamflows and lake levels are considered in combination, an additional flood prone area, shown shaded, is included. Such an exercise is extremely valuable when there is considerable fluctuation in lake levels due to the shape and depth of the lake and prevailing wind direction and there is a flat flood plain and gentle stream slope. Such a combination may not be common, but Southern Ontario streams draining into Lakes St. Clair and Erie fit the above description.

Since peak lake levels and peak river discharges are generally short-term events, the probability of both events occurring simultaneously is small. Again, a mathematical procedure for calculating the joint probability of these two events is extremely difficult. A solution may involve a continuous simulation of instantaneous lake levels and river discharges.

This is an aspect of flood frequency analysis that requires further development.

SUMMARY

There is a growing need for water level frequency information on the Great Lakes shoreline. Procedure for calculating lake level frequencies differ considerably from those used on riverine situations, Until recently very little analysis has been done on lake level frequencies. A procedure for calculating lake level frequencies has been described in this paper. Further research and development is needed on the calculation of flood levels due to the interaction of lake levels and river flows.

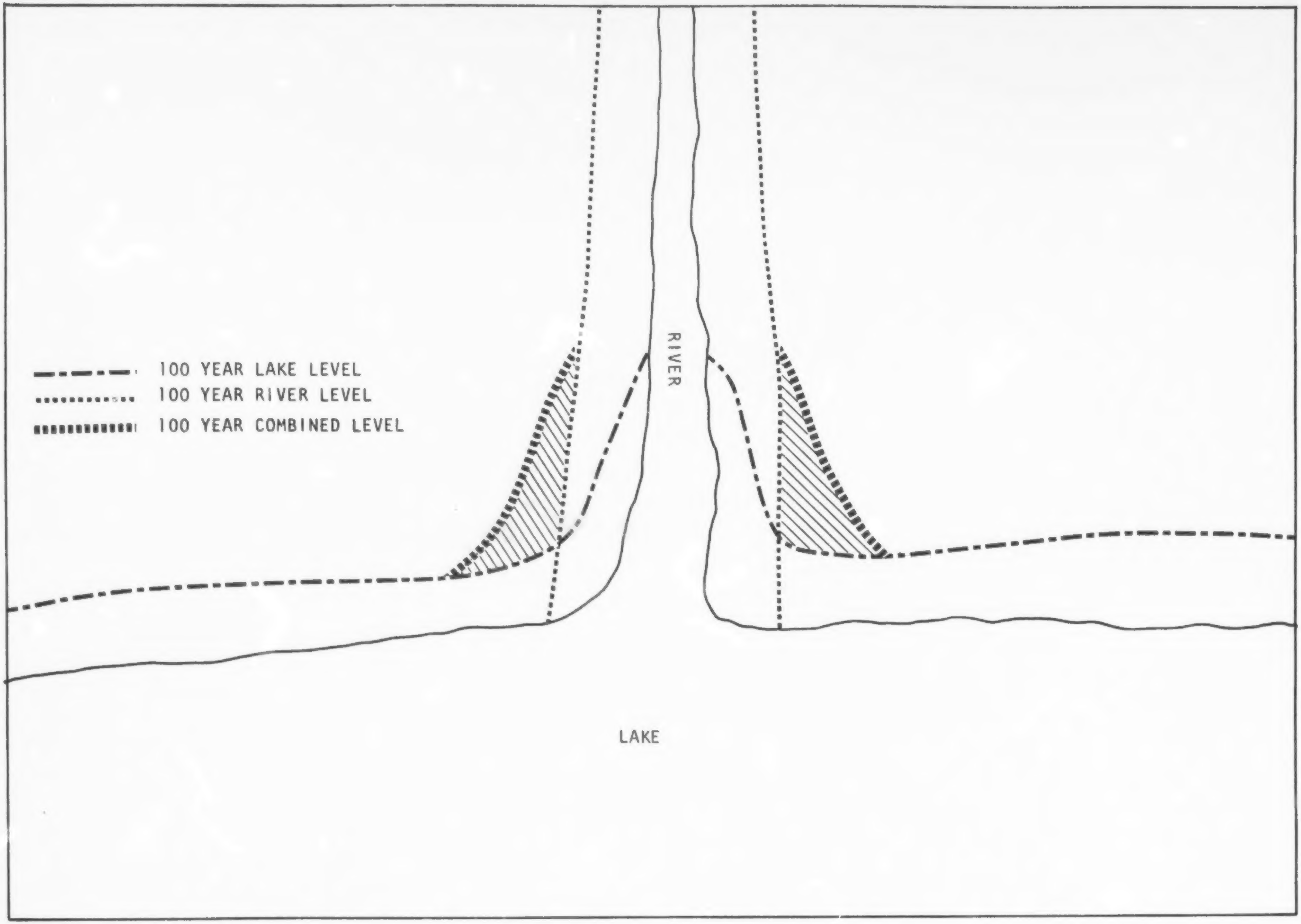
100 YEAR LAKE LEVEL
100 YEAR RIVER LEVEL
100 YEAR COMBINED LEVEL
RIVER
LAKE

Figure 2 Interaction of Lake and River Flood Levels

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